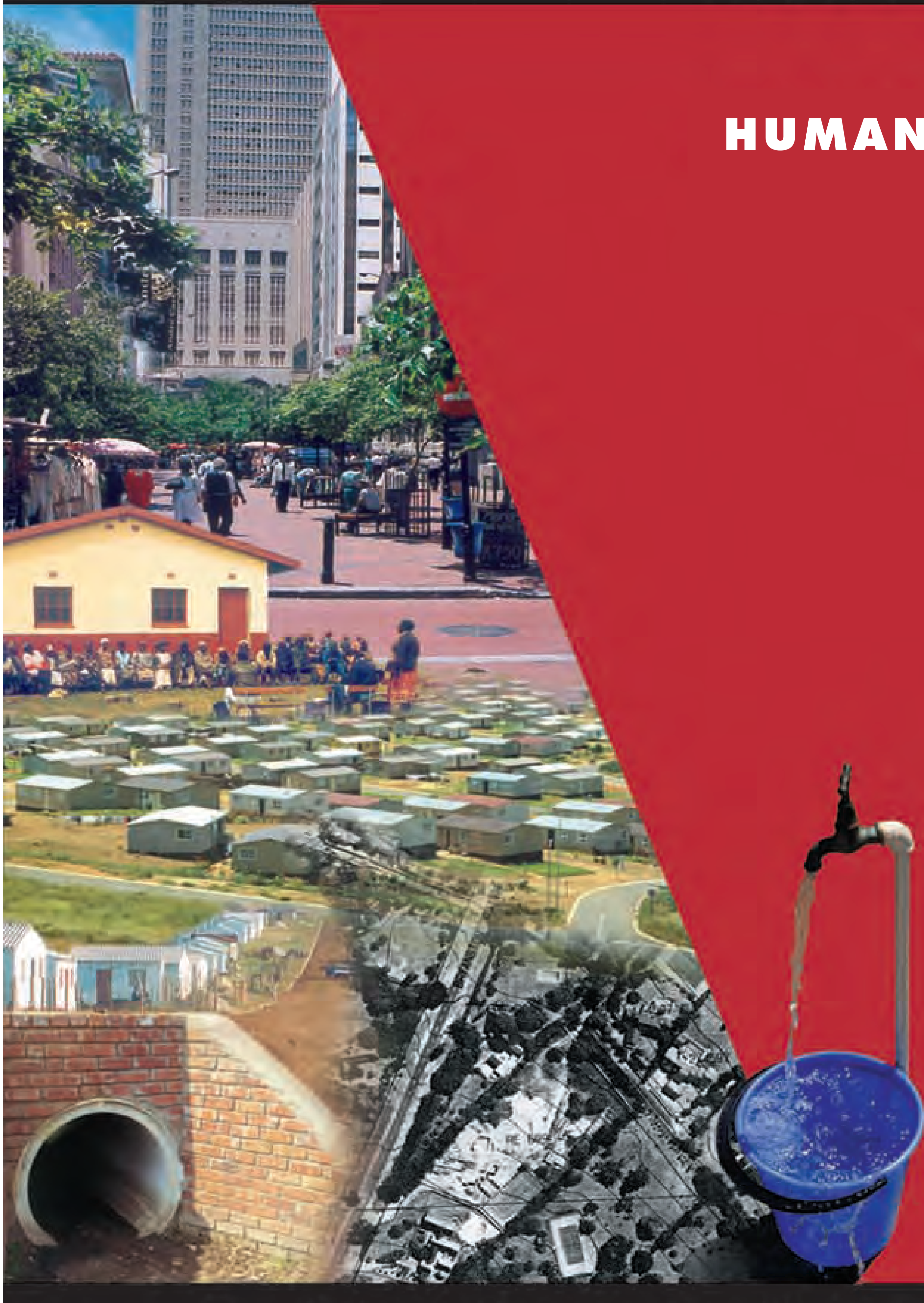
Guidelines for

HUMAN SETTLEMENT PLANNING AND DESIGN

VOLUME 2



Compiled under the patronage of the Department of Housing

by CSIR Building and Construction Technology

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FOREWORD

The establishment of economically, physically, environmentally and socially integrated and sustainable built environments is one of the most important factors which will contribute to harnessing the full development potential of South Africa and addressing distortions of the past and the future needs of our growing population. This goal cannot be achieved without the active participation of especially local government, the private sector and communities in partnership with one another.

This manual, *Guidelines for Human Settlement Planning and Design*, provides a guiding vision for South African settlement formation, addressing the qualities that should be sought after in our human settlements, and providing guidance on how these can be achieved. The publication has been developed over a period of more than two years through a participative process in which stakeholders and experts from various disciplines were involved.

This book is intended to be a living document and you, the reader, are one of its architects. I therefore encourage you to use it, discuss it and debate the guidelines it contains. Still, this work is not the last word on the subject, and your feedback and comments would be welcome. Your active involvement will be the key to the successful attainment of sustainable, habitable living environments in South Africa.

MS S MTHEMBI-MAHANYELE
MINISTER OF HOUSING

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August 2003	9	Water supply	1	-	All
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Index

Volume 1

Volume 2

Stormwater management

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The impact of development on the natural environment

Development is a process of growth and change, which implies improvement. Any development will therefore affect or impact on its environment in some way or other. We consider the building of roads, the erection of buildings and the general improvement of factors that cause inconvenience - like the drainage of stormwater - as development. However, this development may significantly change the hydraulic properties of an area. Typically, pervious layers are rendered less permeable or even impermeable. Depressions are raised to prevent ponding. Surfaces and conduits are constructed to drain runoff more efficiently. Natural vegetation is often removed, allowing reduced interception and transpiration. Limited vegetation cover exposes the soil to the impact of rain, which may lead to increased erosion. Natural meandering watercourses may be canalised to more effectively route flows through the development. Stormwater management is the science of limiting these negative impacts on the environment and enhancing the positive impacts, or catering for the hydraulic needs of a development while minimising the associated negative environmental impacts.

Drainage laws

Increasing development densities have influenced the servitude required to safely discharge runoff into the natural environment. This densification and modification of undeveloped land has also resulted in increased quantities and concentrations of flow with a concomitant increase in pollution.

Upstream landowners' responsibilities for discharging runoff onto downstream properties, and the concomitant responsibilities of downstream owners have a long history which is based largely on common law. This has been modified somewhat by legislation giving certain rights to central, regional and local authorities.

Three rules are generally applicable throughout the world today as far as the drainage of surface runoff is concerned. These concern:

- the "common enemy" concept;
- natural flow; and
- reasonable use.

Stormwater runoff is considered a *common enemy* and each property owner may fight it off or control it by retention, diversion, repulsion or altered transmission. The focus of the common enemy rule has two focal points:

- The need to make improvements to property, with the acknowledgement that some damage results from even minor improvements; and
- The principle of granting each landowner as much freedom as possible to deal with his land essentially as he sees fit.

The *natural flow (or civil law)* rule places a natural easement upon the lower land for the drainage of surface water along its natural course, and the natural flow of the water may not be obstructed by the owner of the lower property to the detriment of the interests of the owner of the higher property.

This rule has been modified to some extent in allowing, for example, surface runoff to be accelerated or otherwise altered into the natural stream. The landowner may, however, neither overtax the capacity of the watercourse nor divert into it runoff that would not naturally have drained into the watercourse (see *Barklie v Bridle* 1956 (2) SA 103 (S.R.)).

In efforts to promote drainage, many of these modifications to the natural-flow rule increase the burden on the lower land.

The *reasonable-use rule* provides that each property owner is permitted to make reasonable use of his land, even though by doing so he may alter the flow of the surface waters and cause harm to others. He incurs liability when his interference is unreasonably harmful (see *Redelinghuis v Bazzoni* 1975 AD 110(T)).

One can see that, in developing property, it is extremely difficult not to concentrate, increase or accelerate stormwater runoff onto downstream properties. Through the provincial ordinances, the authorities have been invested with the right to change the natural drainage of stormwater in the interests of the public as a whole. However with this right comes the responsibility to act with due care in keeping the effects of such deviations within acceptable limits. The general rule that "statutory authority when constructing a work is excused from liability for damage thereby caused to third persons" is subject to the proviso that the work must not be negligently executed or maintained (see *New Heriot Gold Mining v Union Government (Minister of Railways and Harbours*) 1916 AD 415, 421; *Johannesburg Municipality v Jolly* 1915 TPD 432; *Herbert Holbrow (Pty) Ltd v Cape Divisional Council* 1988 (1) SA 387(c)).

To ameliorate this development phenomenon, certain strategies and technologies are available. The goals of stormwater management should support the philosophy of lessening the impact of stormwater flow through and off developed areas. Stormwater should be considered a resource (see Figure 6.1).

Figure 6.1: San Antonio's famous main riverwalk loop. This tourist attraction can be isolated by its sophisticated flood attenuation and flood prevention facilities during major storm events. The project will enable a 100-year flood to be accommodated through the San Antonio River system

It is within the power of the local authority to construct works such as streets and drains which will have an effect on the flow (quantity, quality and velocity) of stormwater discharged on the downstream land.

Public bodies have been entrusted with statutory powers to enable them to carry out public duties. If it is impossible for them to carry out their duties without infringing upon the rights of others, it may be inferred that the legislators intended them to have the power to do so, in spite of the prejudicial effect on individuals. If, however, damage that could reasonably have been prevented is caused to individuals, the failure to take reasonably practicable preventive measures is negligence.

Case law offers opinions and provides principles on which to base the extent of liability of a local authority for damage caused by an increase in stormwater flow. In some cases the issue may be whether interference with private rights is justified where the exercise of statutory powers is alleged to have resulted in an injury to another.

Prohibition against pollution is addressed in Clause 19 of the National Water Act, Act 36 of 1998. This section deals with pollution prevention, and in particular the situation where pollution of a water resource occurs or might occur as a result of activities on land. The person who owns, controls, occupies or uses the land in question is responsible for taking measures to prevent pollution of water resources.

The dual drainage system

Developed areas are defined as any man-induced developments which have changed the environment. In this chapter, all developments in the continuum from rural to urban settings will be addressed.

Traditionally, runoff from frequent (minor) storms has been carried in the urban formal drainage systems. Typically this was achieved by draining runoff from properties into the streets and then via conduits to the natural watercourses. The system was intended to accommodate frequent storms and associated runoff. Today, the value of property is of such significance that engineers need to consider not only frequent storms but the more severe storms, which can cause major damage with sometimes catastrophic consequences (see Figure 6.2). The dual system incorporates a *minor system* for the frequent storm events and a *major system* for the less frequent but severe storm events. The major system may include conduits and natural or artificial channels, but would commonly also make use of the road system to convey runoff overland to suitable points of discharge. This is not very different from what has happened *de facto* except that formal cognisance is now given to the routing of runoff from all storms via the secondary use of roads and other facilities in the urban environment.

The use of the road system and open spaces (such as

Figure 6.2: Damage from major floods can be catastrophic

parks and sports fields) as drainage components of the major system, while imposing inconvenience to road users, is considered an acceptable land-use for these severe storm events.

Development changes the environment

High regard needs to be given to natural drainage patterns and systems. This is because development interferes with these systems. Stormwater management must therefore consider how development has interfered or will interfere with the natural systems. The design engineer, with the planning team, must then plan on how to cope with these changes, to lessen the impact of the altered runoff caused by this development.

It is recognised that development impacts negatively on the natural drainage systems in several ways:

- Permeability of the development area is decreased through increased population densities and fewer open spaces such as parks and gardens, or by the introduction of impervious areas such as surfaced streets, houses and amenities associated with the urban environment. This increases the runoff from the area during storm events, because of the reduced infiltration properties of the development area.

- The introduction of efficient stormwater drainage systems to deal with the *common enemy* implies that the runoff must be conveyed as efficiently as possible to the natural watercourses. The operative word *efficient* is here related to *cost-efficiency*. This has the effect of decreasing the time runoff takes to reach the natural watercourses. The result is a reduction of overland flow, meandering watercourses and the like, through a system which drains runoff to the watercourses as quickly as possible. The flood problem is therefore transferred downstream. Quicker responses in larger catchments make them more susceptible to the effects of high-intensity, shorter duration storm events.

- In more efficient drainage systems, peak flows occur more quickly. This effect has made developments more susceptible to shorter, more intense storm events which in the smaller catchments may lead to greater peak flows.

- The drainage systems are exposed to flows from more frequent, higher intensity storms because of the decreased times of concentration. It is recognised that short-duration storms have higher rain intensities than the longer rainfall events. This increases the pressure put by the frequent, high-intensity storms on the man-made drainage systems, which in turn put more pressure on the natural drainage systems.

- The quality of the runoff deteriorates. One only has to consider runoff from man-made environments which conveys pollutants such as fertilisers, discarded rubbish, spillages and discharges from vehicles, septic tank effluent as well as eroded soil.

The requirement for integrated planning

Against this background, the responsibility for sound planning to lessen these negative impacts rests with the whole design team, and not just with the drainage engineer. A holistic approach to planning needs to be taken, whereby land use is identified and a common commitment to its optimisation forms the background to the planning premises.

Where development projects are carried out with limited funds, the question of “minimum allowable standards” always comes to the fore. It is in these cases, especially, that the development equation becomes a question of balancing low capital cost at development stage against high maintenance costs for the rest of the time.

THE PURPOSE OF STORMWATER MANAGEMENT

Stormwater management is based on

- the need to protect the health, welfare and safety of the public, and to protect property from flood hazards by safely routing and discharging stormwater from developments;

- the quest to improve the quality of life of affected communities;

- the opportunity to conserve water and make it available to the public for beneficial uses;

- the responsibility to preserve the natural environment;

- the need to strive for a sustainable environment while pursuing economic development; and

- the desire to provide the optimum methods of controlling runoff in such a way that the main beneficiaries pay in accordance with their potential benefits.

While these goals may be reflected in other disciplines - and indeed may even be in apparent conflict with one another - specific objectives supporting these overall goals need to be identified for each specific project by the planning team.

PLANNING

Introduction

Planning is a fundamental function of the project team. It determines what the team wants to attain and how it should go about doing so. Some strategic issues need to be considered before any detailed design work can be entertained. This is strategic planning or "master planning".

Master planning

The whole purpose of planning is to facilitate the accomplishment of a project's objectives. Which objectives or goals drive the process really depends on the state of mind of those formulating them, but generally should spell out the why, where, what, when and how of the endeavour. Land use planning should be a consideration of what land resources are available and what they are suitable for, both in the short term and longer term.

Master planning should be concerned with the following principles:

Sustainable development

A key concept of master planning should aim at an enduring or sustainable development.

An initiative called *Caring for the earth, a strategy for sustainable living*, launched in partnership with the World Conservation Union, the United Nations Environment Programme and the World Wide Fund for Nature, stressed two fundamental requirements for sustainable agriculture, namely:

- securing a widespread and deeply held commitment to an ethic for sustainable living; and
- integrating conservation and development.

The World Health Organisation's *Global Strategy for Health and Environment* was a direct result of the United Nation's *Agenda 21* which was drawn up during the Earth Summit in June 1992 at the United Nations Conference on Environment and Development (UNCED) in Rio de Janeiro, Brazil. *Agenda 21* was an action plan to guide national and international activities to ensure that the natural resources of the world are managed in such a manner that sustainable development is achieved. The term "sustainable development" can be defined as development that meets the needs of the present without compromising the ability of future generations to meet their own needs.

The idea that should guide all planning is that there is an interrelationship between the three concepts of health, the environment and development. Any change taking place in the one has a direct influence on the other two.

An essential element of rational land-use planning is land evaluation, which should be a systematic way of looking at the options available and of considering the environment in which these options are likely to operate, so that the results of different courses of action can be predicted. Physical planning therefore cannot be divorced from social and economic circumstances or from administrative and constitutional processes. More importantly, the protection of the environment against pollution decay is essential, to optimise the benefits of sustainable development.

Integration with other disciplines in the planning team

Development produces waste products. When people choose to live closer both to one another and to economic opportunities, there is inevitably an increase in the generation of waste products. The increased volume of wastes needs to be managed to lessen the impact on the environment. Solid waste technologies, the siting of cemeteries, water purification works, sewage works and industries are typical issues facing the planning team in the initial stage. All concern the stormwater specialist.

Layout planning and transport routes inevitably affect the natural drainage of stormwater. These constraints must be explored. Natural habitats will be affected and their importance must not be ignored.

Drainage of both sewage and stormwater is a natural constraint to development that may have to take priority over other services and amenities.

Policy impact assessment

All planning must start with certain premises. Many of these are dictated by outside influences and their impact on the planning at hand must be recognised and considered.

Policy guidelines from external sources may influence the objectives of any development. A policy impact assessment may be required to evaluate the importance of these policies and how they can affect or impact on the planned development. External influences may include policies from

- the World Health Organisation;
- the United Nations (UNCED);

- the South African Constitution;
- the Government's Reconstruction and Development Programme;
- white papers involving water law, water supply and sanitation - Department of Water Affairs & Forestry (DWAF); and
- integrated environmental management procedures - Department of Environmental Affairs & Tourism (DEAT).

While these issues may not all be obligatory, they do serve as a platform or frame of reference from a global and national perspective. They may influence the planning premises.

Strategic impact assessment

Certain constraints to planning and development have to be considered. These are mainly connected with the legal environment. A strategic impact assessment must involve consideration of the aims of the following legislation:

- National Water Act Water Act (Act 36 of 1998):
  To provide for the fundamental reform of the law relating to water resources and to provide for matters connected therewith.

  The legal responsibilities of the stormwater specialist influence the planning. The insertion of 20-year and 50-year flood lines on certain township plans is an example. What development takes place within these flood lines must be explored. The protection of sewage treatment works, cemeteries and solid waste sites from flooding must be considered.

- Conservation of Agricultural Resources Act (Act 43 of 1983):
  To provide for the control over the utilisation of the natural agricultural resources in the Republic in order to promote the conservation of the soil, the water resources and the vegetation and the combating of weeds and invader plants.

  Government Notice No. R1048 of Government Gazette No. 9238 of this Act indicated that:
  No land user shall ... cultivate any land on his farm unit within the flood area of a water course or within 10 metres horizontally outside the flood area of a water course.

  The "flood area" in relation to a water course is defined as *the area which, in the opinion of the executive officer, is flooded by the flood water of that water course during a 1-in-10 year flood*.

- Environment Conservation Act (Act 73 of 1989):
  To provide for effective protection and controlled utilisation of the environment.

- National Roads Act (Act 54 of 1971):
  To provide for the construction and control of national roads, including the disposal of stormwater on a national road.

- Minerals Act (Act 50 of 1991) and its Regulations:
  Specific issues relating to the Environmental Management Programme (EMP) are relevant.

- Minimum requirements for disposal of waste:
  Minimum requirements issued by the Department of Water Affairs & Forestry regarding the selection of sites, their development and management and eventual closure. Three specific references are:

  - *Minimum requirements for the disposal of waste by landfill.*
  - *Minimum requirements for the handling and disposal of hazardous waste.*
  - *Minimum requirements for monitoring at waste management facilities.*

- Health Act (Act 63 of 1977):
  To provide for measures for the promotion of health of the inhabitants of the Republic.

- Atmospheric Pollution Prevention Act (Act 45 of 1965):
  To provide for the prevention of pollution of the atmosphere.

  Specific measures are required for the purification of effluents discharged from appliances for preventing or reducing to a minimum the escape of any noxious or offensive gases escaping into the atmosphere, and for preventing the release of noxious or offensive constituents from such effluents into drains and drainage canals.

- Common law, case law and statutory law:
  Certain laws of parliament, provincial ordinances and government notices can alter existing rules and lay down the law, as it were. These statutory laws create legal duties upon specific persons or bodies and thus determine who is to be sued in delict when damage is caused (Committee of State Road Authorities 1994). Any person or body performing tasks in pursuance of statutory authority bestowed upon him or her to construct works such as streets and drains should heed the principles of the delict, to avoid legal liability for damage suffered by another party.

The basic starting point in South African law as regards the question of damage is the rule that harm rests where it falls, i.e. everyone must bear the damage he suffers, himself (Neethling et al 1990). However, there are grounds which can cause this burden of damage to shift to another, with the result that such other is obliged to bear the former's damage. This shifting (which forms the basis of the law known as law of obligations) of the burden can result from the causing of damage to a person by means of a delict (law of delict). Delict can be defined as the positive act or omission of a person, which in a wrongful and culpable (intentional or negligent) way causes harm to another. The main source of law upon which the South African law of delict has developed is case law.

Other requirements may include:

- Interim guidelines on safety in relation to floods. Safety evaluation of dams. South African Committee on Large Dams, September 1986. The new requirements are contained in Government Gazette Regulation Gazette No 3979, Vol 253, No 10366. Government Notice No R.1559.

- A guideline for groundwater protection for the community water supply and sanitation programme, 1995.

- South African Natural Heritage Programme.

- Conservation of wetlands (Department of Environmental Affairs & Tourism) and reports to the Ramsar Convention.

Stormwater management master drainage plan (MDP):

Master drainage planning should be contemplated on a catchment-wide basis, irrespective of urban and other man-made boundaries. The full environmental impact of the stormwater on that catchment must be investigated and is the responsibility of the controlling regional or local authority. The hydrological processes in the specific area need to be investigated and statistical data obtained. Hydraulic routing of the stormwater must be considered. In analysing stormwater drainage, consideration may need to be given to the use of open spaces like parks, sports fields, and transport circulation routes.

It is assumed that with development there is an increase in both the overall quantity and the peak flow rate of the runoff. Policies in the USA, for example, desire to restrict post-development peak factors to pre-development peak factors. This involves the retarding of stormwater on its route to the drainage streams. The important issue that the drainage specialist must address is the consideration of every storm event - from a severe, infrequent storm event (termed a major event) to the frequent, common storm events (termed minor events). Stormwater drainage technologies must be developed to deal with all these events. A typical formal drainage system avoids the nuisance which might result from frequent storms. This is termed the minor system. The major system will be supported by the minor system but will accommodate the unusually high runoff from infrequent hydrologic events.

Master planning is predominantly concerned with the major system. The minor system will be considered as a supporting one. Master planning may involve

- determination of the recurrence interval of the major flood event;

- determination of the recurrence interval for the minor flood events;

- provision of overall guidelines on runoff-detention requirements, pollution-abatement strategies, and the powers and responsibilities of developers and authorities within the catchment area;

- consideration of land use within flood plains and multi-use of stormwater facilities;

- guidelines on safety and maintenance;

- guidelines on environmental conservation; and

- reference to integrated environmental management (IEM) procedures and principles underpinning the Reconstruction and Development Programme.

While the values in Tables 6.1 and 6.2 are guidelines, the onus is on the drainage engineer to determine the risk associated with a certain recurrence interval. For areas where the risk of monetary loss, loss of revenue or loss of utilities is unacceptably high, a more stringent (or higher) recurrence interval and a higher level of service may need to be considered. On large structures such as bridges and major culverts for example, the Department of Transport has its specific analysis requirements (Committee of State Road Authorities 1994).

A 100-year recurrence interval flood line is required in terms of the National Water Act on residential development plans. Municipal authorities may, however, stipulate other flood lines. The concepts

Table 6.1: Design flood frequencies for major systems

LAND USE	DESIGN FLOOD RECURRENCE INTERVAL
Residential	50 years
Institutional (e.g. schools)	50 years
General commercial and industrial	50 years
High value central business districts	50 - 100 years

Table 6.2: Design flood frequencies for minor systems

LAND USE	DESIGN FLOOD RECURRENCE INTERVAL
Residential	1 - 5 years
Institutional (e.g. schools)	2 - 5 years
General commercial and industrial	5 years
High value central business districts	5 - 10 years

of best management practices (BMPs), good practice, or best available technology not entailing excessive cost (BATNEEC) may convince the planning team that more stringent recurrence intervals need to be considered, including planning for regional maximum floods (RMFs) or probable maximum floods (PMFs).

Detailed design

Stormwater drainage plan

Detailed planning refers to the planning of developments within a catchment. The design philosophies and critical concerns and issues will have been addressed in the master drainage plan. Stormwater drainage planning is the implementation of those policies and guidelines. It should strive to commit responsibilities to the design team and the authorities. Generic responsibilities include

- planning, feasibility studies and preliminary design;
- records of decisions and the development of an auditing system;
- detailed design;
- implementation or construction;
- the process of handing a scheme over to a responsible authority;
- operation and maintenance; and
- monitoring and reporting (elements of the auditing system).

Typically the controlling authority should be responsible for ensuring that detailed planning is compatible with the master drainage plan and that the objectives of stormwater management are attained. The design team needs to consider responsibilities throughout the design life of the project or development - the cradle-to-grave approach.

Responsibilities for runoff control

Stormwater management within an urban area is the responsibility of the local authority for that area, since the control of stormwater is considered a purely local matter.

Certain central and provincial government legislation, however, has encroached on the local authority's planning. Examples are

- the requirement to insert the 1-in-100 year flood lines on all township development plans (National Water Act 36 of 1998);
- prevention of water pollution (consumers) regulated by the departments of Water Affairs & Forestry, Environmental Affairs & Tourism, and Health;
- safety of dams (approved Professional Engineer);
- alteration of a public stream (Transvaal Road Ordinance 22 of 1957; Agricultural Holdings (Transvaal) Registration Act No 22 of 1919; Orange Free State Roads Ordinance 4 of 1968);
- auditing systems and records of decision (IEM); and

- Mines and Works Act.

Detaining and retaining stormwater

Runoff can be stored in constructed dams. However, to be effective, such dams usually demand much space. Successful detention of runoff may therefore have to rely on several technologies involving

- detention ponds (detention facilities) or roof-top detention;
- a preference for overland flow as opposed to hydraulically efficient engineering conduits;
- maintaining pervious surfaces and reducing impervious structures; and
- maintaining vegetation cover to increase interception and evapotranspiration.

MANAGING THE IMPACT OF DEVELOPMENT ON THE ENVIRONMENT

Rural development

The issues and concerns in rural environments may differ from those in the urban setting. Typical issues may include erosion, and soil salinisation. Transport routes may have gravel surfaces, and technologies used to drain runoff need to be carefully considered.

Layout planning

Many of the grid-type layouts inevitably concentrate stormwater in certain roadways. This is usually the case when planners design systems applicable to modern technological standards when they know that upgrading to these standards may take place only in the very distant future, if at all.

What is proposed is a move towards what agricultural engineers and conservationists have always practised - contour planning (e.g. contour ploughing, which is ploughing that follows the contours of land, perpendicular to its slope). This technology is particularly applicable to the more rural type of development (see the section on unsurfaced roads below for more detail).

An example of a tea estate demonstrates this type of layout well (see Figure 6.3). All access paths (which could be substituted by access roads in an urban environment) follow the contours. Other tracks are routed at right angles to the contours so that no extraneous runoff crosses or drains onto them. All stormwater can then be routed along the access paths to the natural channels or waterways.

Figure 6.3: Contour planning: An effective stormwater management tool

The advantage of contour technology is that all flow is routed overland in open channels. This minimises both the flow volumes (drainage occurs regularly) and velocities (open channel flow with flat gradients). This reduces erosion. Soakaways, permeable strata, vegetation interception, retention/detention facilities are some of the many tools at the drainage engineer's disposal. Overland flow in ditches or swales should be designed at the outset with the road layout planners, not afterwards. Where road crossings are required, causeways and drifts are easily constructed. This allows for the use of local labour and materials. Drop structures may also be included with silt traps, allowing maintenance workers to reclaim soil lost from the upstream area.

Preserving the natural environment

Natural resources

Before any layout planning should begin, detailed information regarding topography, geology, hydrology, and fauna and flora, needs to be obtained. Areas containing building materials (sands, aggregates, road materials, clays and the like) should not be sterilised by other land uses without consideration being given to their potential utilisation. Sandy areas are useful for recharging ground water. High-intensity development can be matched with soils of low permeability and vegetation of poor quality.

Alternative fuel sources may need to be developed. Otherwise uncontrolled denudation of the natural landscape may seriously damage the environment, leading to rapid degradation and soil erosion.

All life forms depend on water and, indirectly, on the soil. These two elements must therefore receive priority in any development planning.

All consequences of man's use of natural resources have to be analysed. In many informal settlements, for example, residents grow crops and keep animals, both for subsistence and informal selling. These agricultural practices must be recognised as part of the urban fabric. However, if agricultural practices are to be accepted, potential problems need to be addressed.

Assessing past failures

Poor farming practices have impacted negatively on the environment (Department of Agriculture 1995). Some of the consequences have been the following:

- Decreasing biodiversity. Natural habitats have been destroyed, leading to a decrease in the abundance and diversity of fauna and flora.

- Overgrazing. Overstocking and limited grazing rotation have resulted in denuded land and a threat of desertification. Selective grazing by animals on certain preferred plant species often leads to encroachment by unpalatable and undesirable plant species. Overgrazing is a particularly endemic problem (see Figure 6.4).

Figure 6.4: Overgrazing exposes the land to rain damage and can result in extensive erosion

- Pollution by fertilisers, pesticides, herbicides and fungicides. Cattle dips and feedlots are potential sources of pollution. Their proper design and management cannot be overemphasised (see Figure 6.5).

Figure 6.5: Runoff from feedlots contains high levels of pollutants

  Inorganic fertilisers are often environmentally costly. They can leach out of the soil and contaminate groundwater and streams. Other consequences of injudicious use of fertilisers can reflect in the build-up of toxicity, acidification and salinisation.

  Pesticides often kill non-targeted and usually beneficial organisms in the immediate application area. Non-biodegradable pesticides can accumulate in the soil and water, with hazardous consequences to animal and human life.

- Soil crusting. Incorrect tillage changes the structure of the soil compaction, resulting in poorer water infiltration: runoff increases and there is a greater risk of erosion.

- Dump sites also need protection (see Figure 6.6).

Figure 6.6: Dump sites must be protected against extraneous runoff

TECHNOLOGIES AVAILABLE TO THE ENGINEER

The hydrologic cycle

There are four basic aspects in the hydrologic cycle of interest to the hydrologist, namely:

- precipitation;
- evaporation and transpiration;
- surface runoff; and
- ground water.

Hydrology is used in stormwater management for the design and operation of hydraulic structures. These could include spillways, highway bridges and culverts, or urban drainage systems (Alexander 1990; Linsley et al 1975; Midgley 1972; Adamson 1981).

Flood routing

Many methods and techniques are available to aid the design engineer in calculating runoff hydrographs and associated flood peaks. The scope of this document does not allow for any detailed dealing with these techniques; reference only is made to them. There are many competent references for adequate coverage (Alexander 1990; Midgley 1972; Rooseboom et al 1981; Pansegrouw 1990).

There are also hydraulic simulations, methods and computer programs available that route stormwater runoff and predict hydrographs at nominated reaches in watercourses and drainage ways, e.g. *Hydrosim* (routing model using the kinematic flood-routing equation), and the *SCS method*.

Routing of storm water through drainage ways and rivers requires the computation of varied flow. Gradually varying flow is normally assumed. The computation to determine the variation in depth of flow with distance is available from many sources and is discussed later.

Flood-line determination

The determination of flood lines is usually based on the routing of stormwater through the water course (drainage way). The capacity of a natural channel (or any channel for that matter) is affected by the interaction of local features and the gradually varying flow profile. The routing problem has been addressed by Bakhmeteff (1932), Chow (1959), Henderson (1966) and French (1994). Their classifications of the flow profiles are well documented. Several computer programs are available to aid in the determination of water surface profiles.

Detention and retention facilities

A detention facility is designed to attenuate runoff, specifically the peak flows experienced in the reaches of a water course. These facilities may be planned on a multi-purpose basis as required by the controlling authority, and can achieve a number of stormwater objectives.

Many municipalities now require in their subdivision regulations that runoff from a development may not exceed pre-development runoff for a particular frequency design flood. This is normally accomplished by the construction of a detention basin or facility. The facility acts as a small flood-control reservoir, which can attenuate the peak of the runoff before it flows downstream.

The sizing and positioning of detention facilities needs to be done on a catchment basis, and should form part of the master drainage plan. The combined effect of discharges from various sub-catchments should be analysed to minimise possible adverse effects on the watercourse under consideration.

All dams need to be registered, classified and evaluated as *dams with a safety risk*, specifically any structure with a wall height greater than 5 meters. The evaluation process must involve an Approved Professional Engineer (APE).

To effectively determine the parameters of such a facility, flood routing the design inflow hydrographs through the storage in the basin with a predictable outflow needs to be done, so that downstream effects can be determined. Typical design data may include

- the relevant hydrographs for each of a range of design flood recurrence intervals or frequencies, taking cognisance of the ultimate possible post-development characteristics of the catchment;

- details of the storage and stage characteristics of the detention basin;
- details of the outlet structures with reference to the discharges from the structures at the various stages;
- structural and geotechnical details of the dam wall with regard to type of wall, materials, filters, founding details, spillway structure, erosion protection and freeboard;
- safety precautions related to floods and other hazards;
- possible recreational use of the facility; and
- maintenance issues, including sedimentation and maintenance of vegetation.

See also: American Society of Civil Engineers 1982; Brink 1979; Gerber et al 1980; Jennings et al 1978; Knight et al 1977; SA Institution of Civil Engineers, 1985.

Programs are available to do the flood routing of an inflow hydrograph through a detention facility to produce the downstream outflow hydrograph (HEC-1, Pond Pack). Many of these use the storage-indication working-curve method or Modified Puls method which is discussed in more detail in Appendix A to this chapter (Golding 1981).

Storage facilities may be purpose-built, designed primarily to attenuate runoff, and are usually in the form of wet (part retention, part detention) or dry (detention) facilities. Certain cities in America use underground tunnels and disused quarries, into which peak flows are routed. The effluent is then pumped from these facilities through purification systems before being discharged into the receiving rivers.

Supplementary facilities, whose primary function is not stormwater attenuation but which have been designed to function in an emergency as a stormwater-detention facility, can be designed in parking areas, sports fields and areas upstream of road embankments.

Outlets at stormwater-detention facilities

Culverts

Flow through culverts is dealt with in many text books. Nomographs are available, for example in US Department of Transportation (1985). Flow through culverts is also dealt with in Chow (1959) and French (1994). Most local authorities, provincial and national roads departments have their own design guidelines, to which the drainage designer should refer.

Flow types are categorised according to inlet control, barrel control or culvert, or outlet control. However, all culverts act to some extent as detention structures, because some stormwater upstream of the culvert accumulates before flowing through the culvert. An accurate assessment of the behaviour of a specific culvert configuration for detention facilities needs to be obtained over a range of flows and headwaters (see Figure 6.7).

Figure 6.7: Multiple-outlet structure of a detention pond

Proportional weirs

The proportional weir has one unique characteristic in that it has a linear head-discharge relationship. Details are provided in Appendix B of this chapter.

Improved inverted V-notch or chimney weir

The inverted V-notch (IVN) weir is a more practical, linear sharp-crested weir. Details are given in Appendix B of this chapter.

Spillway crests

Overflow spillway crests are widely used as outlets from detention facilities. Their design is outside the scope of this publication. Reese and Maynord (1987) provide an adequate reference.

Other control structures

Weirs are structures widely used in hydraulic engineering to control or measure flow. Typical weirs include the following:

- Broad-crested weirs
  - Rectangular
  - Triangular

- Sharp crested weirs
  - Rectangular
  - V-Notch
  - Cipoletti weir
  - Proportional or Sutro weir

- Parshall flume.

There are also other weirs. However, there are limitations to the applications of these types of weirs and contractions. French (1994) and Chow (1959) provide ample discussion on the subject. See also Keller (1989) and Francis (1969).

Bridge backwaters

Backwaters produced by bridge restrictions need to be analysed when flood lines are considered. For further reference see US Dept. of Transportation (1978), *Hydraulics of bridge waterways*, Hydraulic design series No 1.

Erosion protection

Energy dissipators

The dissipation of energy of water in canals or of water discharging from pipes must be considered when downstream erosion and scouring are possible. The drainage engineer is always faced with the problem of achieving this in an aesthetically pleasing way. Several technologies are available:

- Widening the drainage way and decreasing the depth of flow. This will have the effect of reducing the velocity of flow. Overland flow is a typical example.

- Increasing the roughness of the canal or drainage way. Although this will increase the total cross-sectional area of flow, it will decrease the velocity.

- Structures which include the following:
  Roughness elements
  USBR type II basin
  USBR type III basin
  USBR type IV basin
  SAF stilling basin
  Contra Costa energy dissipator
  Hook type energy dissipator
  Trapezoidal stilling basin
  Impact-type energy dissipator
  USFS metal impact energy dissipator
  Drop structures
  Corps of Engineers stilling well
  Riprap basins.

Reference to their selection and hydraulic design is given in US Department of Transportation (1983).

Examples of some energy dissipators are shown in Figures 6.8, 6.9 and 6.10.

Figure 6.8: Simple drop structures in a residential environment

Figure 6.9: Energy dissipator

Figure 6.10: Energy dissipator in a multiple land use setting

Structural elements

Structural elements that are typically used include:

- Geocells (e.g. Hyson-cells);
- Geotextiles (e.g. Bidim, MacMat, Sealmac);
- Geomembranes (HDPE linings and others);
- Riprap;
- Gabions (refer to SABS 1200);
- Reno mattresses (refer to SABS 1200);
- Linings (e.g. Armorflex); and
- Stone pitching (adequately covered in SABS 1200).

The reader is urged to approach the suppliers of these products for technical information.

Transitions

Kerb inlet transitions

Kerb inlets (lateral stormwater inlets) are widely used with kerbs and surfaced roads. Their performance has been discussed adequately by others. Forbes (1976), for example, proposed that on moderate to steep road gradients, the capacity could be substantially improved by incorporating an extended length of depressed gutter upstream of the inlet.

Culvert transitions

Culvert transitions are structures that attempt to converge wide, shallow subcritical flows into high-velocity critical flows which can be passed through deep, narrow throats that are more cheaply constructed as culverts or bridges. Sometimes termed *minimum energy* or *maximum discharge* designs, this concept allows large flows to be routed through smaller, more efficient and economical culverts or bridges without the usual backwater or headwater required to provide the energy necessary to pass the flow through a typical opening (Cottman and McKay 1990). Consideration must be given to the immediate downstream effects and energy dissipation.

Modification of the headwall and culvert opening details to conventional culverts and bridges can also reduce the energy loss at the entrance. This lends itself to a more efficient hydraulic design (US Dept. of Transportation 1985, *Hydraulic design of highway culverts*). However, a more efficient hydraulic design through the culvert structure generally leads to higher-energy waters at the outlet. Energy dissipators may have to be incorporated into the design.

Kerb inlets

The standards used by municipalities vary considerably. Generally, cognisance should be taken of the following:

- hydraulic performance;
- accessibility for cleaning purposes;
- ability of the top section of the culvert to bear heavy traffic;
- safety for all road users; and
- cost.

For further information, refer to Forbes (1976).

Side weirs

Side weirs are structures often used in irrigation techniques, sewer networks and flood protection. A special case of this is the kerb inlet (Hagner 1987; Burchard and Hromadka 1986).

Road drainage

Surfaced roads

The main function of urban roads is the carrying of vehicular, cycle and pedestrian traffic. However, they also have a stormwater management function. During minor storm events, the two functions should not be in conflict. During major storm events, the traffic function will be interrupted, the flood control function becomes more important and the roads will act as channels. Good road layout can substantially reduce stormwater-system and road-maintenance costs.

A well-planned road layout can significantly reduce the total stormwater-system costs. When integrated with a major system this may obviate the need for underground stormwater conduits. A key element is that the road layout should be designed to follow the natural contour of the land (Miles 1984). Coordinated planning with the road and drainage engineers is crucial at the pre-feasibility stage.

Stormwater runoff may affect the traffic-carrying capacity through:

- sheet flow across the road surface;
- channel flow along the road;
- ponding of runoff on road surfaces; and
- flow across traffic lanes.

Refer to Figures 6.11, 6.12, 6.13 and 6.14.

Figure 6.11: A lined open roadside channel

Figure 6.12: Incorrect vertical alignment has caused stormwater to bypass this kerb inlet. Kerb inlets in the intersecting road on the left have also not captured runoff from that road so that stormwater flows across this bus route, creating a hazard

Figure 6.13: This ponding is the result of encroachment of the grass verge onto the road. Grass-cutting maintenance operations should also include harvesting the grass, to prevent siltation and build-up of new grass growth

Figure 6.14: Flow across this intersection has resulted in siltation, which is inconvenient and hazardous to road users

Sheet flow

This flow is generated on a road surface and is usually least at the road crown, increasing towards the road edge. This can lead to hydroplaning when a vehicle travelling at speed has its tyres separated from the road surface by a thin film of water.

Sheet flow can also interfere with traffic when splashing impairs the vision of drivers.

Channel flow

Channel flow is generated from sheet flow and from overland flow from adjacent areas. As the flow proceeds it increases in volume, encroaching on the road surface until it reaches a kerb inlet or drain inlet. The result is reduced effective road width. Again, splashing produced by the tyres can lead to dangerous driving conditions.

It is important that emergency vehicles should still be able to use the road during major storms.

Figures 6.15 and 6.16 illustrate examples of channel flow.

Figure 6.15: Example of surface drainage in a residential setting

Figure 6.16: Open channel alongside an arterial road

Ponding

Ponding on roads may occur at low points, at changes in gradient, at sump inlets and road intersections. This can have a serious effect on traffic flow, particularly as it may reach depths greater than the kerb height or remain on the roadway for long periods. A particular hazard of ponding is that it is localised and traffic may enter a pond at high speed.

Flow across traffic lanes

Flow across traffic lanes may occur at intersections, when the capacity of the minor system is exceeded. As with ponding, localised cross-flows can create traffic hazards. Therefore, at road intersections, for example, traffic devices should be used to reduce traffic speed during downpours. In such cases allowing cross-flow may be preferable to maintaining the road gradient, which would have the effect of creating irregularities in the cross slope of the road.

Encroachment on roads by runoff

Major storm events

The encroachment by runoff from a major storm event onto primary roads should not exceed a depth of 150 mm at the crown of the road. This will allow access by emergency vehicles.

Minor storm events

The suggested maximum encroachment on roads by runoff from minor storms is given in Table 6.3.

Road gradients

Maximum road gradients

The maximum road gradient should be such that the velocity of runoff flowing in the road edge channels does not exceed 3 m/s at the limits indicated in Table 6.3. Where the velocity of flow exceeds this value, design measures should be incorporated to dissipate the energy.

Minimum road gradients

The minimum gradient for road edge channels should be not less than 0,4% (to reduce deposition of sediment).

Maximum road crown slope

The maximum slope from the crown of the road to the road edge channel is not governed by stormwater requirements.

Table 6.3: Suggested maximum encroachment of runoff on roads during minor storms

ROAD CLASSIFICATION	MAXIMUM ENCROACHMENT
Residential and lower-order roads	No kerb overtopping.* Flow may spread to crown of road.
Residential access collector	No kerb overtopping.* Flow spread must leave at least one traffic lane free of water.
Local distributor	No kerb overtopping.* Flow spread must leave at least one lane free in each direction.
Higher-order roads	No encroachment is allowed on any traffic lane.

* Where no kerb exists, encroachment should not extend over property boundaries.

Minimum road crown slope

The minimum slope from the road crown to the channel should be not less than 2% for a plain, surface or an average of 2% where the surface has a variable cross slope.

Unsurfaced roads

The drainage function of unsurfaced roads is dependent - and has a significant impact - on the planning of the road and access layout. If the roadway is to be used to channel and drain stormwater runoff, then the velocity of this runoff must be such that minimal erosion potential exists. This means that gravel roads, together with their side drains, need to have low gradients. Roads with steep gradients should, as far as possible, not be used as drainage ways, nor should any adjacent side drains without proper erosion protection. This protection can include drop structures, lined channels at critical sections, or regular drainage from the roadway into intersecting roads or drainage ways. Refer to Figures 6.17 and 6.18.

Figure 6.17: The contour-planning concept, to impede the drainage of runoff from the development

Figure 6.18 (a): Concentration of stormwater drainage in a roadway resulting in major inconvenience to road users

Figure 6.18 (b): Concentration of stormwater drainage in a roadway resulting in major inconvenience to road users

Runoff from earth or gravel roads will contain grit and its conveyance in pipes can eventually block or even damage the pipe network. Blockages are difficult to clear. In many instances maintenance is not done, rendering the network ineffectual. Use of pipelines in environments of high erosion potential is not recommended because of the high expense of maintenance and high risk of failure or non-performance.

An alternative to a network of pipes is an open-channel system on the roadsides for conveying minor storm runoff. This system may also convey dry weather flow in areas where there are high or perched water tables, and there is sullage from individual households or from communal water points. The positioning of communal water points must be carefully considered and soakaways or drainage from these points included in their design (see Chapter 9). Suitable erosion protection of the canal invert, including drop structures and silt traps, is important. See Figure 6.19

Figure 6.19: Roadside drains of the major system

The use of open roadside channels may necessitate a wider road reserve than that required to accommodate subsurface drains. This is particularly pertinent where open channels intersect with roadways or property access ways. The width of an open channel may increase progressively as the drain accepts more runoff. The road reserve may have to be widened or the channel deepened. The advantages of these alternative technologies must be compared. Open drains, like all systems, will require maintenance. However, this technology is not "out-of-sight, out-of-mind". Siltation and other problems will immediately become apparent.

Where the whole roadway is used as a drainage way for the major system, erosion protection on the downstream road edge may need to be considered. The crossfall of the road should generally be against the natural ground slope so that the whole road width can act as a drainage way in the major system. To maximise the storage function of the roads as part of the drainage system in major storm events, the settlement layout should be planned so that the greatest length of road closely follows the ground contour (the contour-planning concept). Figure 6.20 shows an example of this planning concept of a rural village in highly erodible granitic soils. The erosion from this area was effectively eliminated.

Figure 6.20 (a): Layout planning in steep terrain

Figure 6.20 (b): Layout planning in steep terrain

Roof drainage

High-intensity short-duration storms are experienced in many parts of southern Africa. These violent storms are often accompanied by strong winds and hail, which may lead to damage and the flooding of buildings containing valuable assets.

It is considered prudent that designers of roof structures for large buildings take cognisance of the roof drainage system. The research done by Schwartz and Culligan (1976) suggested that the five-minute storm is likely to cause overtopping of gutters in buildings of conventional size. The approach to the design of roof lengths, gutter size, box receivers and downpipes is discussed in this reference.

In the United States of America certain authorities require that all runoff generated on certain properties be stored for slow discharge into the municipal drainage system. Ponding of rain on roofs is one retention technology. Others include using depressed areas on the property to store excess runoff. See Figures 6.21 and 6.22.

Figure 6.21: This office complex incorporated the entrance to the building in a detention facility. Under normal conditions, this area functions as a pleasant park with water features

Figure 6.22 (a): Depressed area to store excess runoff

Figure 6.22 (b): Runoff from this parking area is stored in the drainage ways using multiple-outlet drains

Upgrading issues

While greenfield development may encompass a more rigid planning procedure, upgrading projects should also be subjected to assessment of their possible environmental impact. In development, where greenfield development may be at the one end of the continuum and projects associated with mature developed areas at the other, the issues to be addressed in planning should be similar. A rigorous assessment of impacts - which may include operation and maintenance issues - should be considered. The classification of project proposals and their possible effects on certain areas and features are considered fully in the Guideline Document 1 of the IEM procedure (Department of Environment Affairs 1992b).

POLLUTION ABATEMENT

While the issue of quantity of stormwater runoff has been addressed in South Africa for some time, emphasis on the quality of runoff has lagged behind. While economic and social attention to development has been largely beneficial, the total cost of technology (that is, the opportunity costs of not considering the total waste stream produced by a technology) has not been considered in great depth in South Africa. This typically refers to issues, like health care, which concentrate on treating the symptoms instead of striving for a balance between prevention (reducing or eliminating the cause) and cure. The user, the man in the street, pays sooner or later, carrying both financial cost and forfeiting quality of life.

Sources of pollution

Air pollution

With the exception of pollution generated by motor transport, stationary fossil-fuel processes produce the bulk of air pollution in South Africa (Petrie et al 1992). Five activities in this category which generate air pollution are recognised:

- fuel combustion and gasification from stationary sources (particularly electrical power generation);
- fossil fuel burning in dense unserviced settlements;
- industrial and chemical processes (ferro-alloy industries, fertiliser production);
- solid waste disposal (incineration of industrial, residential and hospital wastes); and
- land surface disturbances (mining and construction activities, agricultural practices and veld fires).

Of particular concern is the acidic deposition by rain (acid rain) which may result in acidification of freshwater ecosystems, denudation of forested and agricultural areas, corrosion of metal surfaces and destruction of masonry structures.

While the Atmospheric Pollution Prevention Act provides for both administrative and judicial control measures, air pollution control is administered in practice by the chief officer of a local authority and the government mining engineer.

Point sources of water pollution

<u>Industrial</u>

Apart from the air pollution produced by industry, most industrial processes use water and produce effluent. Some of these processes include

- abattoirs;
- breweries;
- pharmaceuticals;
- fishing;
- tanning; and
- the fruit and vegetable industry.

The strategies for pollution abatement are focused on the reduction of water use at source and improved effluent purification technologies.

Runoff from industrial plant sites may contain toxic or hazardous pollutants. The installation of detention facilities can be one technology to store the effluent during periods of runoff and release it at a slower rate to a treatment process such as a wetland.

<u>Mining</u>

There are many pollutants emanating from mining operations, depending on the operations themselves. Probably the main concern to the drainage engineer is the so-called acid mine drainage and heavy metals.

The standard practice on South African mines varies considerably. The fundamental principles of keeping clean water and polluted water separate and preventing clean water from becoming contaminated are largely ignored by many mines (Pulles et al 1996).

Non-point sources of water pollution

Non-point sources of water pollution are difficult to locate. Some success has been obtained using infra-red aerial photography (Perchalski et al 1988). This technology can prove helpful in identifying non-point source problems that include

- oxygen depletion in dams; and
- runoff from city streets, farms, forests, mines, construction sites and atmospheric deposition.

<u>Agricultural pollution</u>

Poor farming practices have often impacted negatively on the environment. Some of the observed consequences have been

- decreasing bio-diversity;
- overgrazing through overstocking;
- pollution by fertilisers;
- pollution by pesticides, herbicides and fungicides;
- soil crusting; and
- irrigation with polluted water.

The scarcity of water has been identified as the single most limiting factor for economic growth in southern Africa. It is therefore essential that this scarce resource be utilised judiciously and sensibly for the benefit of all users, including the natural environment.

Certain standards of water quality are required for all users - from primary domestic use to water for irrigation, stock watering, recreation and the maintenance of aquatic habitats.

Water used for irrigation will always contain measurable quantities of dissolved salts. These include relatively small but important amounts of dissolved salts originating from the dissolving or weathering of rocks, soil, lime, gypsum and other salt sources as the water passes over or percolates through them. The suitability of water for irrigation will be determined by the amount and type of salts present. While the quality does not form part of the scope of this chapter, poor water quality may cause various soil - and therefore productivity - problems to develop, namely

- salinity;
- permeability;
- toxicity; and
- miscellaneous chemical problems (e.g. excess nitrogen in the water, pH).

Guidelines in terms of the sodium adsorption ratio (Ayers 1977) and the adjusted sodium adsorption ratio should be consulted to ensure that productivity is not limited by the water quality, and that users downstream are not disadvantaged by poor agricultural land use.

Urban pollution

Stormwater runoff from urban catchments has been found in many areas to be a major source of pollution to the water-receiving bodies. Practices such as effective rubbish collection, street cleaning, vegetation buffer strips and improved fertilizer practices ameliorate this pollution (see the section on street cleaning in Chapter 11: Solid waste management).

Soil erodibility

Examples of highly erodible soils in South Africa include the granitic soils found in Mpumalanga and the Highveld (Kyalami system). Although erosion is a natural phenomenon, interference by man with the natural environment can rapidly degrade the natural systems available to ameliorate this erosion. One of the major anthropological influences which occurs in many of the rural areas is the over-stocking of animals. See Figure 6.23.

Dispersive clays occur in any soil with high exchangeable sodium percentage (ESP) values. The testing procedures for the identification of dispersive clays are discussed by Elges (1985). Methods for constructing safe and economic structures with dispersive clays should be carefully considered.

Figure 6.23: Gully erosion by headwater upstream of a highway culvert inlet. Specific attention needs to be paid to structures founded in soils with high erosion potential

Grass-lined channels

Grassed waterways are generally a cost-effective way to convey stormwater (US Dept Agriculture 1987). The use of indigenous plants for stabilisation is recommended. The velocity of water flowing in the waterway should be limited in relation to the erodibility and slope of the waterway.

Fencing off the waterway in a rural environment is usually an effective way of controlling livestock so that the grass cover can be established. Once this cover has been established, livestock can be introduced onto the grassed waterway in a controlled manner. Figure 6.24 illustrates the concept.

Figure 6.24: An example of a grassed waterway

Detention ponds

While authorities in the USA stipulate detention times for certain areas, the general use of this technology to settle out suspended solids and for the treatment of other pollutants from runoff in South Africa appears remote, as the incoming runoff usually has a high clay content. Siltation occurs in all dams. However, from a purification point of view, it is not considered economically feasible in detention facilities to settle out the products of erosion (i.e fine silts and clays). Detention times would need to be very long, and therefore require large areas of land. Retention facilities should rather be considered for their beneficial use as water bodies and integrated with the natural drainage corridors which allow other natural processes to occur (e.g. wetlands).

Wetlands

Background

Walmsley (1987) defined wetlands as "water dominated areas with impeded drainage where soils are saturated with water and there is a characteristic *fauna* and *flora*". Other items such as *vleis*, water sponges, marshes, bogs, swamps, pans, river meadows and riverine areas are often included in references to wetlands. Their ecological importance has often been emphasised, and many municipalities and industries are using artificial wetlands to treat waste water (Gillette 1993).

Wetlands purification functions

The principle function of the vegetation in wetlands is to create additional environments for microbial populations. The stems and leaves in the water obstruct flow, facilitate sedimentation and increase the surface area for the attachment of microbes, and constitute thin films of reactive surfaces.

Wetland plants also increase the amount of aerobic microbial environment in the substrate. This is incidental to the unique adaptation of wetland plants to thrive in saturated soils. Most terrestrial plants cannot survive in waterlogged soils for any appreciable length of time due to the oxygen depletion which normally follows flooding. Unlike their terrestrial relatives, wetland plants have specialised structures which enable them to conduct atmospheric gases, including oxygen, down into the roots. This oxygen leaks out of the root hairs, forming an aerobic rhizosphere around every root hair, while the remainder of the subsurface water volume remains anaerobic. This juxtaposition of aerobic/anaerobic environments is crucial to the transformation of nitrogenous compounds and other substances.

Microbial organisms

Microbes (bacteria, fungi, algae and protozoa) alter contaminant substances in order to obtain nutrients or energy to complete their life-cycles. The effectiveness of wetlands in water purification is dependent on developing and maintaining optimal environmental conditions for the desirable microbial populations. Most of the desirable microbes are ubiquitous and are likely to be found in most waste waters with nutrients and energy; hence inoculation of specific strains is generally not necessary.

Substrate

The primary role of the substrate, whether soil, gravel or sand, is to provide a support for the plants and a surface for attachment by microbial populations. The substrate can also be selected on the basis of its chemical characteristics, where pollutant removal is achieved through complexing - a chemical and physical process allowing more complex substances to be formed, which then remain in the substrate zone (Batchelor 1993).

Plant selection

The diversity and complexity of natural wetland vegetation is principally the result of interactions between the following three important factors:

- hydrology;
- substrate; and
- climate.

In constructing a wetland - if the intention is to attempt to reproduce a "natural" wetland - a thorough knowledge of local wetland vegetation is a distinct advantage. If, however, the intention is to use the wetland for water purification, there are a number of universal species that can be used. By far the most commonly used genera are *Phragmites*, the common reed and *Typha* (referred to locally as the bulrush). Other genera that have been used successfully are various *Scirpus* and *Cyperus* species.

Advantages of constructed wetlands

Constructed or created wetlands simulate processes occurring in the natural wetland system. These include

- flood attenuation (if properly designed); and
- water-quality improvement through the removal of substances such as suspended solids, nitrogen, trace metals, bacteria and sulphates.

These are only a few of the substances wetlands are able to remove. Their effective functioning is dependent on the pressure of environmental conditions conducive to the required processes. They are not dependent on external energy or chemical inputs. Wetlands also require little maintenance.

In addition to their water purification functions, wetlands provide habitats and life-support systems for a wide range of flora and fauna, particularly birds, plants, reptiles and invertebrates. Wetlands can be aesthetically pleasing and offer an opportunity for recreation and education. See Figure 6.25.

Figure 6.25: Part detention, part retention facility, including an artificial wetland

Protection of the environment during the construction phase

Erosion protection

One of the most unstable periods in any development occurs during the construction phase. Runoff from sites should be collected in temporary check dams (see below). Straw bales can be positioned at kerb inlets to prevent silt entering the underground drainage systems while construction is taking place. Figures 6.26 and 6.27 illustrate the examples of temporary erosion protection.

Figure 6.26 (a): Construction sites can be prevented from polluting the surrounding area by the use of straw bales, mulching and geo-mats. Aggregates placed at exits from sites prevent the transport of pollution

Figure 6.26 (b): Construction sites can be prevented from polluting the surrounding area by the use of straw bales, mulching and geo-mats. Aggregates placed at exits from sites prevent the transport of pollution

Figure 6.27: Stone aggregate can be placed on the roadside while the shoulder grass establishes itself

Protection and rehabilitation

The erosion cycle - detachment, transportation, deposition - is a natural phenomenon. However, development usually alters the natural pattern. Erosion control is a measure to substantially reduce erosion and thereby decrease the sediment input into the stream system. The aim is to reduce the accelerated erosion typically caused by poor agricultural techniques and other land uses. Measures may include practices such as contour farming and terracing (applicable to both the rural and urban settings), strip cropping, crop rotation, no-till farming, grassing drainage ways, gully and stream bank erosion control, and stabilisation of critical areas.

Technologies available to the designer include the following:

Silt fencing and straw bale barriers

Silt fences and hay/straw bale barriers are two types of filter barriers. They are temporary structures which are installed across or at the toe of a slope. They are used to control sheet flow and are therefore not effective in areas of concentrated flow, such as ditches or waterways. See Figure 6.28.

Figure 6.28: The straw-bale barrier has allowed pioneer plants to establish themselves

Temporary check dams

Small temporary check dams constructed across a ditch or small channel reduce the velocity of concentrated stormwater flows. They also trap small amounts of sediment. Temporary check dams are useful on construction sites, or for temporary stabilisation of erosion areas where protection is required for the establishment of vegetation.

Riprap

Riprap is a heavy stone facing on a shorebank used to protect it and the adjacent upland against wave scour. Riprap depends on the soil beneath it for support and should be built only on stable shores and bank slopes.

Other technologies

The use of geofabrics, matting, netting, mulching and brush-layering are other technologies which attempt to protect the soil from rain impact and impede the flow of stormwater runoff.

The alternative biological approach is often integrated with the structural technologies. Many of the indigenous flora can be effectively used to form vegetation buffer strips and other natural barriers, sponges and stable riverine corridors (Oberholzer 1985).

Street cleaning

Street cleaning can be an effective method of removing litter and sand-sized particles. Overall pollutant removal by street sweeping is not, however, very efficient, with an upper limit of 30 per cent removal being reported in the USA for well-run programmes and a more typical removal of 10 to 30 per cent (Sartor et al 1984).

Organics and nutrients are not effectively controlled, but regular - daily or twice daily - street cleaning can remove up to 50 per cent of the total solids and heavy metal yields in urban stormwater (Field 1985).

Runoff from unmanaged urban environments can be highly polluted. Efforts to reduce this pollution must be coordinated between those responsible for refuse removal, sanitation, and industrial effluent.

Waste disposal sites

The siting of waste disposal sites is regulated by the minimum requirements guidelines published by the Department of Water Affairs & Forestry (1994a, b and c). For the drainage engineer, the importance of siting relates to the environmental impact the sites may have on the ground and surface water bodies. As waste disposal sites are regarded as a bulk service, they are not discussed further in this document, except to illustrate the adverse effects of uncontrolled or unmanaged dumping (see Figures 6.29 and 6.30).

Figure 6.29: Unmanaged domestic refuse is a serious health risk

Figure 6.30: Uncontrolled dumping at municipal waste sites can have a serious impact on surface and sub-surface water systems

Sanitation

Waterborne sanitation

Illegal stormwater drainage into sewer gulleys is endemic throughout South Africa. The increase in flow into sewage works after storm events is widely recognised (Institute of Water Pollution Control 1973). While no estimates of flow are suggested, one empirical estimation for increased flow under wet weather conditions is equal to one and a half times the peak dry weather flow. However, because of the variations observed in many sites, it is advisable to place gulleys away from where stormwater flows or collects, and gulleys should be as few in number as practicable.

On-site sanitation

The protection of the environment from possible pollution by on-site sanitation systems such as pit toilets and soakaways is crucial. Pollution may be caused by infiltration of the leachate from the pits into the groundwater, or by surface runoff through a sanitation system which is positioned in a surface-water drainageway. The preservation of groundwater resources (many South Africans use wells, springs and boreholes) is particularly important. In this regard, recommendations of the Department of Water Affairs & Forestry (1995) are relevant.

Water supply

In developing communities, consider the drainage of excess water from water points. Erosion can occur (see Figure 6.31).

Figure 6.31: This water point has been placed in a low-lying area. Drainage of runoff and water spilled from the point has not been allowed for, and a health hazard has resulted

Selection of cemetery sites

The following ten criteria are deemed essential (Fischer 1992) when planning the position of a cemetery:

- soil excavatability for ease of excavating the graves;
- soil permeability which affects the pollutants' ability to be transported into groundwater;
- the position with respect to domestic water supplies, specifically if groundwater resources are being used;

- the position with respect to drainage features, so that stormwater runoff does not convey pollutants to streams or pose a threat of exposing graves. The position of the 1:50 floodline or a more stringent floodline should be part of the investigation;

- site drainage, which influences the ingress of rain into the soil and therefore the graves, with the possibility of pollutants then being conveyed from the site through the soil;

- site topography, which influences access to graves as well as surface and subsurface drainage;

- basal buffer zone, which relates to the depth of soil (buffer) between the deepest grave and the water table (permanent or perched);

- grave stability relates to the stability of the soil around the grave from the time of excavation to the time of interment;

- soil workability, which refers to the ease of compaction; and

- cemetery size, which relates to the requirements of future grave sites and development pressures.

Cemetery pollution

According to Fischer (1992) cemeteries pose a pollution threat in that many of the existing cemeteries in SA contaminate our water resources. Microbiological pollutants (including bacteria, viruses and parasites) remain active within the water table at much greater distances from their source than was previously assumed.

At present no legislation exists to control the planning of cemeteries in South Africa. Linked to the issue of land availability, the potential of pollution from cemeteries requires a thorough technical evaluation.

MAINTENANCE ISSUES

Detention facilities

The nature of problems encountered with a stormwater facility depends on its type, function, location and general environment. Many of the problems can be avoided by proper design and construction procedures. Table 6.4 lists some of the problems commonly experienced with these facilities.

The control of weed growth and invader plants, and the mowing of lawns is necessary for aesthetic and health (mosquito control) reasons. To ensure that the bottom of a dry pond remains dry, it should be sloped (typically at 5 per cent) to the outlet or to a sub-channel linking the inlet to the outlet. The sub-channel may confine silt and enable storm runoff from minor events to bypass the pond. Subsurface drainage may be required to prevent swampy conditions when a surface channel cannot be conveniently installed.

Aquatic weed growth can be reduced in wet (retention) ponds by designing the ponds to have a minimum depth of water of 1,2 meters (after allowing for sedimentation). Aeration and disinfection of the water may also be necessary to maintain the required quality. The proper design of grids over outlets (limiting the opening to about 314 mm^2) assists in reducing outlet blockages. Reliable emergency spillways that cannot block should be provided. The pollution of wet ponds, where the initial runoff cannot be routed to bypass the facility, can be reduced to some extent by installing grease and sediment traps upstream of the pond inlet.

However, reduction or elimination of the pollutants at their source is generally the preferred option.

The failure of a facility's embankment may result from piping, either because of poor soils (e.g. dispersive soils), inadequate filter designs, animals which burrow into the embankment, or tree root systems. Legumes should not be used as plant cover for earth embankments as this results in a concentration of nitrogen in the roots, which attracts rodents.

When sports fields are used as detention facilities, playing surfaces are generally raised slightly above the surrounding area to facilitate drainage and clearing of general debris and siltation. See Figure 6.32.

Figure 6.32: These sports fields will act as a detention facility should the watercourse on the right of the picture not be able to convey the flood. Note that the playing surfaces are raised to aid drainage after such an event. The crest of the side weir between the watercourse and the sports fields is designed to accept the excess flow, but is made aesthetically pleasing with a meandering cycle path

Where porous pavements are used in parking areas or roads to encourage infiltration or reduce traffic noise,

Table 6.4: Problems commonly experienced with storage facilities (after APWA Special report No 49 1981, and DeGroot 1982)

PROBLEM TYPE	COMMENTS
Weed growth	Easy access to the site will enable the maintenance department to combat weed growth. Designers should establish acceptable pioneer grasses (e.g. *Cynodon dactolon* - fine kweek, *Stenotaphrum secundatum* - buffalo grass) and reeds and marsh plants where applicable.
Maintaining grass	Bank slopes should be gentle enough to allow access by maintenance equipment.
Sedimentation and urban litter	Increased maintenance required, but confined to a specific site.
Mosquito control	Regular mowing required - keeping grass short facilitates evaporation and provides access for predators of mosquito larvae.
Outlet blockages	Particular detail required for outlet works and may incorporate straw bale filters and trash racks.
Soggy surfaces	Landscape the basin so that depressions can be utilised as retention areas for artificial wetlands.
Inflow water pollution	Regular maintenance during wet season. Consider wetland filters at inlets.
Algal growth	Attempt to create a sustainable ecology.
Fence maintenance	Consider public access points to the facility.
Unsatisfactory emergency spillway design	Dam design requirements.
Dam failures and leaks	Dam design requirements (e.g. dispersive soils).
Public safety during storm events	Flood warning systems are an integral part of the total design.

the surfaces should be regularly cleaned of debris to prevent clogging.

Design details

Minimum pipe diameters

The suggested minimum pipe diameters are

- 300 mm in a servitude; and
- 375 mm in a road reserve.

Minimum velocities and gradients in pipes

The desirable minimum velocity range is 0,9 - 1,5 m/s. Low velocities will not prevent siltation, so the maintenance of pipe networks needs to be considered at the design stage (see Table 6.5).

Anchor blocks

Concrete anchor blocks (20 MPa concrete strength) should be provided, as in Table 6.6.

Figures 6.33 to 6.36 illustrate some common maintenance problems with stormwater facilities which can be avoided by careful planning and design.

Table 6.5: Suggested minimum grades for pipes (City of Durban 1984)

PIPE DIAMETER (mm)	DESIRABLE MINIMUM GRADIENT (1 IN ...)	ABSOLUTE MINIMUM GRADIENT (1 IN...)
300	80	230
375	110	300
450	140	400
525	170	500
600	200	600
675	240	700
750	280	800
825	320	900
900	350	1 000
1 050	440	1 250
1 200	520	1 500

Table 6.6: Suggested spacing for anchor blocks (City of Durban 1984)

GRADIENT (1 IN...)		SPACING FOR 2,44 m PIPE LENGTHS
2	(50%)	every joint
2 - 3,33	(50% - 30%)	alrternate joint
5	(20%)	every 4 th joint
10	(10%)	every 8 th joint

Figure 6.33: Inlet grids need attention

Figure 6.34: Kerb inlets require maintenance

Figure 6.35: The use of underground drainage systems in unpaved roadways should be justified

Figure 6.36: Stormwater drainage must occur off the roadway if kerbing is not installed

ENVIRONMENTAL HEALTH AND SAFETY

The role of education and community involvement

Wood consumption for cooking and heating purposes can be greatly reduced by the use of efficient stoves (Ellis 1987), in lieu of open fires. Less wood consumption leads to less denudation of the landscape.

The establishment of woodlots, although a long-term project, will help alleviate the wood shortage. Again, these woodlots must be protected and managed, so the communities having access to the resource must have the necessary institutional capacity to do this.

Veld fires are a common occurrence during the winter months in many areas in South Africa. No land-user may burn veld or allow burnt veld to be grazed without the written authorization of the executive officer (Forestry Act No 72 of 1968). While it is generally agreed (Booysen and Tainton 1984) that veld burning (typically every 3 - 5 years) improves the quality of the veld, regular yearly burning reduces the basal cover of the grass and therefore subjects the soil in the area to rain beat and possible erosion. Not only does the burning pollute the atmosphere, but resources such as thatching materials (*Hyparrhenia filipendula*, *H. dissoluta* and *H. hirta*) may be lost. Although grasses usually recover before the rains, when burned in the middle of winter, fires in early spring may expose the soil to potential erosion. A preferred technology is mowing; this, however, is more expensive.

Awareness of hazardous situations and flood-warning systems

Flood-warning systems are an integral component of the design of any hydraulic structure, and are a local authority responsibility. Stormwater management should aim to eliminate flood hazards.

Effective forecasting and warning of impending flooding will enable a community to prepare and therefore reduce the danger to life and property. Flood warnings are usually more effective in large catchments, where long response times allow intervention by the authorities and the public. However, local areas subject to flood hazards can be signposted and safety measures incorporated into the land-use design. (See Figure 6.37.)

Figure 6.37: Flood-warning systems

Recreational use of facilities

Watercourses need to be managed and maintained in urban environments. Flood plains are particularly valuable facilities for indigenous fauna and flora. One particular maintenance problem is the management of alien invader plants (see Figure 6.38). Maintenance by the relevant authorities may place a high burden on the funding available for general parks and recreational maintenance. The commitment to maintain these natural corridors should be communicated to all involved and not be the burden of the authorities only. Many campaigns (Collect-a-can, for example) encourage public participation in maintaining our natural areas.

Figure 6.38: This nature trail has a problem with pampas grass (Cortaderia jubata), an alien invader

General precautions for development on dolomites

Figure 6.39: Dolines are almost always the result of man's intervention in natural drainage systems. This sinkhole has been caused by a road culvert

The following recommendations have been made by the Council for Geoscience (Calitz 1993):

House structures

- All man-made ponds, drainage ways, stormwater conduits and road surfaces should be made impervious.

- Runoff from structures should be conveyed in impervious conduits away from the structures. If gutters are required by the local authority, the downpipes should discharge into a lined or precast furrow. This furrow should discharge the water 1,5 m away from the from the foundation, preferably onto a grassed ground surface sloping away from the structure. Where no gutters are utilised, it is recommended that an apron 1,5 m wide be provided along the external walls of the structure where water will discharge from roofs. This will allow the runoff from the roof to be distributed away from the structure foundation.

Installation and maintenance of services

- Bulk services should be routed in road reserves or servitudes with a minimum width of five meters.

- Sewer- and water-reticulation systems in the road reserves or other servitudes. If these services are placed mid-block, a building line restriction of a minimum width of 5 m should be imposed. Place water and sewer connections of any two erven on their common property boundary. Shared sewer connections should be considered if this arrangement leads to a reduction of the length of piping and/or minimises the disturbance to the environment.

- Each erf should have a rodding eye or similar access point to the sewer connection, in addition to an inspection eye.

- All stormwater, water and sewer pipes must be watertight.

- Avoid the use of clay pipes. When using polyethylene (HDPE) or polyvinyl chloride (uPVC) pipes, the use of compression-type joints is preferred. The use of heat fusion, glue fusion or mechanical bonding requires good workmanship and may rupture with small differential movements in the soil.

- The roots of trees planted in close proximity to the line of water-bearing services often cause leaks. Due regard should be taken when positioning trees and other plants.

- Trenches and excavations, once opened, should be closed as soon as possible. Proper compaction of the backfilling is recommended to obviate the trench acting as a soakaway drain.

- The excavated material may be unsuitable for use as pipe bedding or initial backfilling material because of the possibility of the presence of coarse chert.

- Water pipes entering buildings should be fitted with flexible couplings or kinked with a Z-form to allow for relative movement.

- Corrodible pipes should be protected.

- Where pipes cross roads, consideration should be given to the use of sleeves or culverts (ease of access for maintenance).

- Once the service has been installed and backfilled, the ground surface profile should be restored to its natural slope.

- All laid drainage pipes and sewerage should be tested for leakage at time of installation and thereafter at regular intervals (CSIR, NBRI Info sheet X/BOU 2034).

See also: Donaldson 1963; Buttrick and Van Schalkwyk 1995; Buttrick and Roux 1993; and Roux 1991.

GLOSSARY: Terms used in stormwater management

- **CUTA**: Committee of Urban Transport Authorities, Department of Transport, PO Box 415, Pretoria 0001.

- **Design period**: Design period is usually the length of time a structure or asset will be expected to have a useful life, or the amortisation period if loans have been procured to finance the construction of the structure or asset.

- **Design storm**: The properties of a storm include the depth, spread and duration of the rainfall as well as variations in rainfall intensity in space and time over the catchment area during the storm. In the choice of a design storm cognisance should be taken of all these parameters.

- **Detention facility**: A structure which temporarily stores excess stormwater for a length of time. The outlet of the structure is designed to release the stored water into the downstream watercourse at a rate less than the flow rate into the facility during storm events.

- **Drainage area**: That part of a catchment above a specified point that contributes to the runoff at that point.

- **Drainage system**: See "stormwater drainage system".

- **Dry pond**: A detention pond that remains dry during dry weather flow conditions.

- **Dry weather flow**: Flow occurring in a water course not attributable to a storm rainfall event, but to groundwater flow where the water table intersects the stream channels of a catchment. These dry weather flows do not fluctuate rapidly because of the low flow velocities.

- **Evapotranspiration**: The evaporation from all water, soil, snow, ice, vegetation and other surfaces plus transpiration of moisture from the surface membranes of leaves and other plant surfaces.

- **Flood plain**: The flood plain of a river is the valley floor adjacent to the incised channel, which may be inundated during high water (Linsley et al 1975). CUTA defines the floodplain as the area inundated in a river by the major flood.

- **Flood plain fringe**: The flood plain fringe is that area in a river defined as being below the level reached by the regional maximum flood and above the level reached by the major flood. Note: The US National Flood Insurance Programme has definitions of "floodway" and "fringe area" that differ from those of CUTA.

- **Flood zone or floodway**: Defined by CUTA as the area inundated by the regional maximum flood (RMF).

- **Groundwater runoff**: That part of the infiltrating water that percolates downward until it becomes groundwater. Eventually, after an indirect passage, it enters a stream that intersects its path and becomes dry weather flow or base flow.

- **Hyetograph**: A graph indicating the distribution of rain relative to time.

- **Hydrograph**: A graph of stage or discharge relative to time.

- **Interception**: Precipitation stored on vegetation as opposed to rain in surface depressions (termed depression storage).

- **Major drainage system**: A stormwater drainage system which caters for severe, infrequent storm events. Supported by the minor drainage system.

- **Minor drainage system**: A stormwater drainage system which caters for frequent storms of a minor nature.

- **Nomograph**: A chart or graph from which, given a set of parameters, other dependant parameters can be ascertained.

- **Off-stream facility**: A stormwater facility which is situated away from the normal drainage way. A typical example is shown in Figure 6.32. The sports fields alongside a drainage way will act as a detention facility should the bank of the drainage way be overtopped by flood waters.

- **On-site facility**: Detention facility located on a watercourse and through which normal dry weather flow passes.

- **Outfall**: See "stormwater outfall".

- **Recurrence interval**: Recurrence interval or return period is the average interval between events. The recurrence interval is usually expressed in years and is the reciprocal of the annual probability. That is, the event having an annual probability of occurrence of 2% (0,02) has a recurrence interval of 50 years. This does not imply that such an event will occur after every 50 years, or even that there will necessarily be one such event in every 50 years, but rather that over a much longer period (like a 1 000 year period) there will very likely be 20 events of equal or greater magnitude.

- **Retention facility**: A structure which retains runoff indefinitely should the capacity of the structure be sufficient to contain such runoff. Excess flow into the structure will be discharged via a spillway.

- **Runoff**: The water which constitutes streamflow may reach the stream channel by any of several paths from the point where it first reaches the earth as precipitation. Water which flows over the soil surface is described as surface runoff and reaches the stream soon after its occurrence as rainfall. Other water infiltrates through the soil surface and flows beneath the surface to the stream.

- **Stormwater drainage system**: All the facilities used for the collection, conveyance, storage, treatment, use and disposal of runoff from a drainage area to a specified point.

- **Stormwater outfall**: The point at which runoff discharges from a conduit.

- **Subsurface runoff**: The flow derived from water infiltrating the soil and flowing laterally in the upper soil strata. It reaches the receiving streams or bodies of water fairly soon after a rainfall event, without joining the main body of groundwater.

- **Surface runoff**: That part of the runoff that travels over the ground surface and in channels to reach the receiving streams or bodies of water.

- **Sustainable development**: Development that meets the needs of the present without compromising the ability of future generations to meet their own needs.

- **Watercourse**: Stream or channel. May or may not have a permanent flow of water.

APPENDIX A

Storage-Indication working curve method or Modified Puls method.

The standard flood routing equation is

$$\left(\frac{I_1 + I_2}{2}\right)\Delta T - \left(\frac{Q_1 + Q_2}{2}\right)\Delta T = S_2 - S_1 = \Delta S \qquad \text{(A1)}$$

This can be transposed to

$$\left(\frac{I_1}{2}\right) + \left(\frac{I_2}{2}\right) + \left(\frac{S_1}{\Delta T}\right) + \left(\frac{Q_1}{2}\right) - Q_1 = \left(\frac{S_2}{\Delta T}\right) + \left(\frac{Q_2}{2}\right) \qquad \text{(A2)}$$

where

I_1 and I_2 are the inflow rates at times T_1 and T_2 respectively;

Q_1 and Q_2 are the outflow rates at times T_1 and T_2 respectively;

S_1 and S_2 are the storage values at times T_1 and T_2 respectively;

ΔT is the time increment between time T_2 and T_1.

In equation (A2) all the values on the LHS are known. Initially I_1, Q_1, S_1 and I_2 are known, yielding $\left(\frac{S_2}{\Delta T}\right)+\left(\frac{Q_2}{2}\right)$. By developing the relationship between Q and $\left(\frac{S}{\Delta T}+Q\right)$, Q_2 can be interpolated from the value $\left(\frac{S_2}{\Delta T}\right)+\left(\frac{Q_2}{2}\right)$ and S_2 can be interpolated from the relationship between Q and S. In the next time step the terms Q_2 and $\left(\frac{S_2}{\Delta T}\right)+\left(\frac{Q_2}{2}\right)$ become Q_1 and $\left(\frac{S_1}{\Delta T}\right)+\left(\frac{Q_1}{2}\right)$ respectively, so that the third storage value and discharge value can be deduced, and so on.

APPENDIX B

Proportional and inverted V-notch weirs

The relationships for proportional weirs are as follows:

$$Q = C_D . \sqrt{2ga} \, . \, b \left(h - \frac{a}{3} \right) \qquad \text{(B1)}$$

where the discharge coefficient C_D can be assumed to be approximately 0,62

and

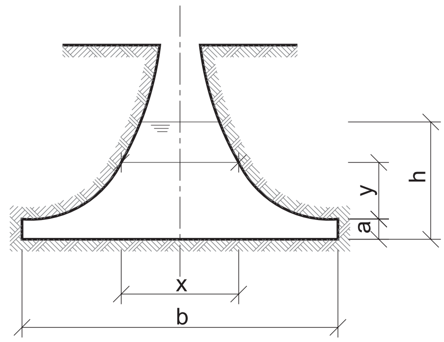
$$\frac{x}{b} = 1 - \frac{2}{\pi} \tan^{-1} \left(\sqrt{\frac{y}{a}} \right) \qquad \text{(B2)}$$

where

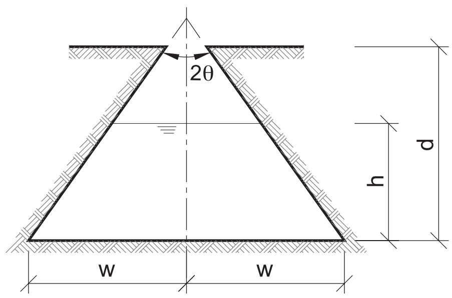
Q is the flow in m^3/s

a, b, h, x and y are in meters and shown in Figure B6.

DOWNSTREAM ELEVATION
PROPORTIONAL WEIR



DOWNSTREAM ELEVATION
INVERTED V-NOTCH WEIR



The relationship is as follows:

$$q = \frac{2}{3} . C_D . \sqrt{2g} \, . \, 2Wh^{3/2} - \frac{8}{} . C_D . \sqrt{2g} . \tan \theta . h^{5/2}$$

for 0 < h < d

For flows through this weir above a depth of 0,22d but less than 0,94d (where d is the V-sloped section height), Murthy (1990) found that the discharges are proportional to the depth of flow. See Figure B6.

Figure B6: Proportional weirs

APPENDIX C

Broad crested weir

The most commonly used formula for estimating flow over a broad crested weir is formula (C1). As long as the flow is critical and parallel along its length, the equation is a good estimate.

$$q = 1{,}70H_2^{3/2} \tag{C1}$$

Where

q = flow per unit width of weir.
H_2 = total specific head at the weir.

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Chapter 7

Roads: Geometric design and layout planning

7

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SCOPE

In this chapter the major objectives are to

- stress the importance of geometric design, giving expression to planning concepts;
- emphasise the revised approach to the road hierarchy;
- suggest the importance of satisfying the needs of all road users, both vehicular and non-vehicular; and
- provide guidelines for detailed geometric design that will result in a safe, efficient, affordable and convenient road and street system.

INTRODUCTION

Reference to planning

The ultimate objective in the creation of an urban place is that it should be such that people would wish to live, work and play there. This can only be achieved by the closest co-operation between the planner and the geometric designer, because the ultimate layout of the street system effectively defines the urban area in terms of its functionality and, hence, its attractiveness to the inhabitants.

These two disciplines must also interact closely with the other disciplines involved in the provision of services to the inhabitants. As streets also form sets of conduits along which essential services such as water supply, sewerage and power are conducted, they should be so located that they do not unnecessarily constrain the provision of these services.

This chapter of the guidelines cannot, therefore, be read in isolation.

Both the planner and the designer are required to adopt a more holistic approach to determination of the street network than has previously been the case. They may find it useful to consider the total width of the cross-section as being hard open space, only part of which is dedicated to the movement function. This part of the cross-section is roughly equivalent to the road reserve as previously understood and is still required to address a range of trip purposes, trip components and modes of travel.

With regard to modes of travel, the design of the street network was historically predicated almost exclusively on the passenger car. Many trip makers will, however, always be reliant on walking or public transport as the only modes available to them. Furthermore, it is not possible to endlessly upgrade the street network in terms of a growing population of passenger cars.

Road design should, therefore, not only accommodate public transport but actively seek to encourage its use. Adding embayed bus stops to a route essentially designed for passenger cars does not constitute support for the promotion of public transport.

Obviously, bus routes should be designed with the bus as the design vehicle. This selection of design vehicle impacts, inter alia, on decisions concerning maximum gradients, lane widths and provision for bus stops. A designated bus route should have a horizontal alignment planned to enhance the attractiveness of the route to would-be passengers and also be highly accessible to pedestrians by ensuring that walking distances to the nearest bus stop are minimised. Dedicated bus lanes should be provided in areas where the volume of bus or other high-occupancy vehicles warrants their use. The dedicated bus lane implies that the street will essentially be a shared facility serving other modes of transport as well. The high volumes of bus traffic that are typically achieved when a number of bus routes converge on the CBD or some other transport hub may suggest that dedicated bus routes, as opposed to bus lanes, become a practical option.

The distinction drawn between *geometric planning and geometric design* is not always clear. In this document, geometric planning is described in Chapter 5: Planning Guidelines. A brief exposition of the difference between planning and design is offered below.

Geometric planning

Planning addresses the broad concepts in terms of which the functions of the various links in the street network are defined. These concepts address the sum of human activity, whether economic (which can be formal or informal), recreational or social - the latter including educational, health care and worship activities. In this context, it is pointed out that movement is a *derived* activity or demand.

Previously, both planning and design tended to focus on areas being dedicated to single land-use, thus forcing a need for movement between the living area and any other. Current planning philosophies favour the abandonment of single use in favour of mixed use. Mixed use suggests that people can both live and work in one area. Not only will this have the practical effect of reducing the demand for movement over long distances but, where movement is still necessary, it will support change of the mode of movement because, over short distances, walking and cycling are practical options. This will further reduce the areal extent required to be dedicated to movement. Conceptual planning leads to the definition of corridors intended to support some or other activity, one of which is movement.

Corridors, in association with their intended functions, will ultimately define the horizontal alignment of the streets located in them. A need for high traffic speeds will suggest high values of horizontal radius, whereas reductions in radius could be applied to force speeds down to match activities involving a mix of vehicular and non-vehicular traffic. Reduction of the lengths of tangents between curves or intersections could serve the same purpose.

The dominant function of the corridor defines the vertical alignment in terms of maximum and minimum acceptable gradients, vertical curvature and length of grade. For example, streets with a predominantly pedestrian function should ideally be flat, whereas - if movement includes provision for a bus route - modest gradients are allowable. Routes intended principally for the movement of vehicles other than buses may be steeper although, where very high volumes are anticipated, the adverse effect of steep gradients must be borne in mind.

Function is defined in terms of two prime components, namely the nature and the extent of demand. As such, it is the major informant of the design of the cross-section. The demand may be for high-speed, high-volume traffic flows, in which case the cross-section would comprise more than one moving lane in each direction, possibly with a median between the opposing flows and shoulders as opposed to sidewalks. On the other hand, if the demand is for predominantly pedestrian/ commercial activity, very wide sidewalks (i.e. wider than would be required merely to accommodate a volume of moving pedestrians) would be necessary to allow for sidewalk cafes, roadside vending and browsing or window shopping. While vehicles would not necessarily be excluded, their presence would not be encouraged and speeds would be forced down by having few and narrow lanes and very short tangent lengths.

Geometric design

Design is principally concerned with converting to physical dimensions the constraints introduced by planning concepts. Ongoing reference to the chapter on Planning is necessary to ensure that the road as ultimately designed matches the intentions regarding its function. It is important to realise that the function of the road reserve is broader than merely the accommodation of moving traffic which may be either vehicular or pedestrian. Although geometric design tends to focus on movement, the other functions must be accommodated. If the designer does not adopt this wider perspective, the most likely consequence would be that the original intention - “the creation of an urban place that should be such that people would wish to live, work and play there” - would be severely compromised.

The goals of transportation as propounded by the Driessen Commission are the economic, safe and convenient movement of people and goods with a minimum of side-effects. These goals are unchanged. It must be understood that, in arriving at an acceptable street design, these goals apply equally to all modes of travel. In this respect, it will invariably be necessary to seek compromises between the various modes. The one goal seen as being non-negotiable is safety. For example, the safety of pedestrians cannot be compromised in pursuit of convenience of vehicular travel.

Classification of the road and street system

The traditional five-level hierarchy of streets has effectively been abandoned, principally because it placed an over-emphasis on the vehicular movement function of the street system. The concept of a hierarchy also implicitly carried with it the notion of one part of the network being more important than another. The network comprises a system of interlinking streets serving different functions, and often serving these different functions differently.

Over-emphasis of the importance of one link at the cost of another does not only constitute poor design; it can place the network as a whole in jeopardy. In fact, all parts of the network require equal consideration. To assist designers in developing some understanding of the new classification system, Table 7.1, offering a comparison between the previous five-tier system, the Urban Transport Guideline (UTG) series, and that currently employed, is shown below.

It should, however, be clearly understood that there is not a one-to-one relationship between the current and the other classifications. The five-tier and the UTG classification systems are limited to addressing movement in terms of a spectrum of accessibility versus mobility, whereas the current classification addresses all functions of roads and streets. Reference should be made to Chapter 5.1: Movement Networks, in which the classification system is comprehensively described.

Mixed routes forming part of the “movement network” classification can be subdivided into “higher-order”, “middle-order” and “lower-order” routes. Higher-order routes would carry higher volumes of traffic *and/or accommodate higher levels of economic activity*, whereas lower-order routes would principally address local and access-seeking traffic and *accommodate higher levels of recreational activity.* Middle-order routes serve primarily as links between higher- and lower-order routes. It would thus be unwise to regard the appellation of “higher-order” as an invitation to reach for, say, UTG 5.

The five-tier system subdivided Class 5 streets into a further six sub-classes, two of which could be loosely

Table 7.1: Comparison between classification systems

MOVEMENT NETWORK	FIVE-TIER SYSTEM	URBAN TRANSPORT GUIDELINES (UTG SERIES)
Vehicle-only route	1 Regional distributor	(Freeways not included)
	2 Primary distributor	Major arterial (UTG1)
Mixed pedestrian and Vehicle route	3 District distributor	Minor arterial (UTG1)
	4 Local distributor	Collector (UTG15)
	5 Access street	Local street (UTG7 &10)
Pedestrian-only route		(not applicable)

classified as being pedestrian-only. The UTG series does not address either freeways or pedestrian-only routes. UTG 10 addresses local commercial and industrial streets which fall outside the ambit of this document.

Measures of effectiveness (MOE)

Geometric design is primarily concerned with the assemblage of a group of components leading to the creation of an operating system. As a case in point, the cross-section is not whole and indivisible: it is, in fact, heavily disaggregated. In the preparation of a design, it is thus necessary not only to be aware of the various functions that the geometric design is intended to serve but also to be able to measure the extent to which the often conflicting functions are served. Invariably, the local authority will specify that the facility provided should meet some or other specified level of utility. Measures of effectiveness (MOE) are thus required.

With regard to movement, MOEs can relate either to management of the operation of infrastructure, or to the provision of infrastructure.

In the former case, reference is to Transportation System Management (TSM) and its strategies and processes, all of which have MOEs associated with them. Reference should be made to *Guidelines for the transportation system management process* (1991), Pretoria: Committee of Urban Transport Authorities (Draft Urban Transport Guidelines: UTG 9). This document is available from the National Department of Transport.

Management is aimed at enhancing the productivity of the system as provided. A well- managed system could, with a lesser extent of infrastructure, conceivably accommodate the same demand for movement as an unmanaged system. By the same token, a design that is sensitive to the possibilities contained in good management would result in a highly efficient use of space.

A lesser extent of infrastructure implies that more land is available for other applications and the capital outlay involved in infrastructure provision per erf is lessened. This benefit applies directly to all aspects of community life. Residential properties are rendered more affordable because the costs incurred in servicing them inevitably impact on property prices. The financial return from commercial properties is enhanced because the lower land price has an effect on the business overheads of the occupier. In short, commercial activities can become more competitive.

MOEs that refer to the provision of infrastructure invariably refer to the Transportation Research Board Special Report 209: *Highway Capacity Manual (HCM)*, the most recent edition of which was issued in 1994. This manual is in general use in South Africa. The manual comprehensively addresses the entire spectrum of modes of movement, and all aspects of the road or street network, from freeways to residential streets to intersections. The HCM propounds a philosophy of Levels of Service (LOS) and, in general, refers to five levels ranging from A to E with LOS A being the highest level and LOS E corresponding to capacity. The higher levels are typically related to LOS E by utilising a volume-to-capacity or v/c ratio. The actual MOEs on which the levels of service are based vary between the various types of facility being analysed. Those applying to a freeway do not apply to a residential street or to a signalised intersection so that, although in all cases reference could be made to LOS A, what is intended is not comparable between them.

It is recommended that analysis be based on the Highway Capacity Manual, with due regard being paid to the practical benefits to be derived from the application of TSM measures.

Traffic calming

Traffic calming refers to measures usually designed to reduce either the volume of moving vehicles or their speed. The intentions behind the application of traffic calming can, however, be many and various. In addition to reductions in speed or volumes of traffic, they may include:

- noise reduction;
- reduction of air pollution; and
- provision of safe areas for pedestrians or other non-vehicular road users.

Planning aimed at achieving these intentions reduces the need for the introduction of traffic calming measures which are essentially artificial devices such as speed humps, chicanes, street narrowing devices, road closures and changes in surfacing colour or texture.

Planning measures could include the selection of link lengths as a means of reducing variations in vehicle speed. As a rule of thumb, a link length in metres that is about ten times the desired speed in km/h would ensure that a vehicle entering the link at less than the desired speed is not likely to accelerate beyond it. Alternating priority control at the intersections along a street would also serve as a restraint on excessive speed, as would bounding the various links by three-legged or T- intersections.

Small-radius horizontal curvature would effectively cause a reduction in traffic speeds. However, this application is not recommended for general practice. Highly curvilinear alignments impact adversely on the costs of provision of all the services normally located within the road reserve. Furthermore, pedestrians would be subjected to unnecessarily long travel paths between origin and destination.

Should traffic calming measures be necessary, an area-wide approach is preferable to isolated measures. The effect is to impose the desired conditions over a wide area so that low speeds or volumes become part of the drivers' expectations in respect of the area being traversed. An isolated speed hump located where a driver does not expect it can result in loss of control or damage to the vehicle itself.

Traffic calming devices are discussed in detail in Schermers G and Theyse H (1996), *National guidelines for traffic calming.*

BASIC DESIGN PARAMETERS

The design vehicle

The design vehicle is a composite rather than a single vehicle. It thus represents a combination of the critical design features of all the vehicles within a specific class weighted by the number of each make and model vehicle found in the South African vehicle population.

The dimensions offered in Table 7.2 were determined by Wolhuter and Skutil (1990). The values quoted in the table are 95 percentile values.

Because of its application in the determination of passing sight distance, the fifth percentile value of height is selected. The height of passenger cars is thus taken as 1,3 m. A height of 2,6 m is adopted for all other vehicles.

Two vehicles are recommended for use in the design of urban roads. The passenger car should be used for speed-related standards and the bus for standards relating to manoeuvrability, typically at intersections. The bus also dictates the maximum permissible gradient. Designs must, however, be checked to ensure that larger vehicles, such as articulated vehicles, can be accommodated within the total width of the travelled way, even though they may encroach on adjacent or even opposing lanes. Should these larger vehicles comprise more than 10% of the traffic stream, it will be necessary to use them as the design vehicle.

In constricted situations where templates for turning movements are not appropriate, the capabilities of the design vehicle become critical. Ninety-five percentile values of minimum turning radii for the outer side of the vehicle are given in Table 7.3. It is stressed that these radii are appropriate only to crawl speeds.

Truck speeds on various grades have been the subject of much study under southern African conditions. Bus speeds are similar and it has been found that

Table 7.2: Dimensions of design vehicles (m) (after Wolhuter and Skutil 1990)

VEHICLE	WHEEL BASE	FRONT OVERHANG	REAR OVERHANG	WIDTH
Passenger car (P)	3,1	0,7	1,0	1,8
Single unit (SU)	6,1	1,2	1,8	2,5
Single unit + trailer (SU +T)	6,7+3,4*+6,1	1,2	1,8	2,5
Single unit bus (BUS)	7,6	2,1	2,6	2,6
Semi-trailer (WB-15)	6,1+9,4	0,9	0,6	2,5

* Distance between SU rear wheels and trailer front wheels

Table 7.3: Minimum turning radii	
VEHICLE	MIN RADIUS (m)
Passenger car (P)	6,2
Single unit (SU)	12,8
Single unit + trailer (SU+T)	14,0
Single unit bus (BUS)	13,1
Semi-trailer (WB-15)	13,7

performance is not significantly affected by height above sea-level. Performance can therefore be represented by a single family of curves calculated on the basis of the 95 percentile mass/power ratio of 275 kg/kW, and as shown in Figure 7.1.

Pedestrians are also considered to be “design vehicles” in the sense that they are self-propelled, occupy space and have a measurable speed of movement. When bunched with others, while awaiting an opportunity to cross a street for example, the individual pedestrian occupies a circular space of about 700 mm in diameter. Pedestrians on the move will, however, prefer to be one metre or more apart. Walking speed on average is 1,5 m/s but in certain areas, such as in the vicinity of old-age homes, hospitals and schools, allowance should be made for lower speeds.

High volumes of pedestrians also invariably force lower walking speeds. Under these circumstances, design should be predicated on a walking speed of 1,0 m/s.

The design driver

Research (Pretorius 1976, Brafman Bahar 1983) has indicated that 95% of passenger car drivers have an eye height of 1,05 m or more, and 95% of bus or truck drivers an eye height of 1,8 m or more. These values have accordingly been adopted for use in these guidelines.

A figure of 2,5 seconds has been generally adopted for reaction time for response to a single stimulus. American practice also makes provision for a reaction time of 5,7-10,0 seconds for more complex multiple-choice situations. These extended times make provision for the case where more than one external circumstance must be evaluated, and the most appropriate response selected and initiated.

The road surface

The road surface has numerous qualities which can affect the driver’s perception of the situation ahead, but skid resistance is the only one of these qualities taken into account in these guidelines.

Skid resistance has been the subject of research worldwide, and it has been locally established by Mkhacane (1992) and Lea (1996) that the derived values of brake force coefficient are appropriate to the southern African environment. Lea established that a limiting value of 0,4 is appropriate to gravel surfaces for all speeds. This suggests that, for design purposes, a value not greater than 0,2 should be adopted for these roads. With regard to surfaced roads, there is a considerable range of values. At 50 km/h the skid resistance of a worn tyre on a smooth surface is half that of a new tyre on a rough surface, and at 100 km/h it is five times lower. Skid resistance also depends on speed, and reduces as speed increases.

The speed used in the calculation of guideline values is the operating speed, generally 80-85% of design speed.

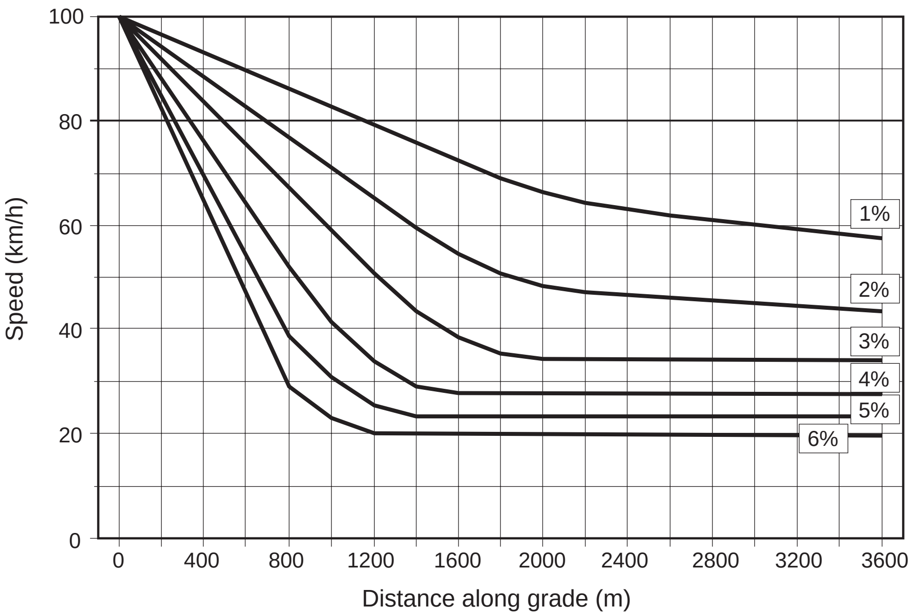


Figure 7.1: Truck speed on grades

Brake-force coefficients are given in Table 7.4. No allowance is made for a safety factor, as these represent actually measured values for a worn tyre on a smooth wet surface which, in engineering terms, constitutes a “worst case”.

Table 7.4: Brake force coefficients

SPEED (km/h)	COEFFICIENTS
20	0,47
40	0,37
60	0,32
80	0,30
100	0,29
120	0,28

THE ELEMENTS OF DESIGN

Design speed

Traffic speeds are measured and quoted in kilometres per hour. The Highway Capacity Manual (Transportation Research Board 1994) lists definitions of ten different speeds, such as spot speed, time mean speed, space mean speed, overall travel speed, running speed, etc. In this document, reference is principally to design speed and operating speed.

The design speed is a speed selected for the purposes of the design and correlation of those features of a road (such as horizontal curvature, vertical curvature, sight distance and superelevation) upon which the safe operation of vehicles depends. The design speed should thus be regarded more in the nature of a grouping of various design standards rather than as a speed per se.

The operating speed is the highest running speed at which a driver can travel on a given road under favourable weather and prevailing traffic conditions without, at any time, exceeding the design speed. Implicit in this definition of operating speed is the idea that the design speed is also the maximum safe speed that can be maintained on a given section of road *when traffic conditions are so favourable that the design features of the road govern the driver’s selection of speed.*

Sight should not be lost of the fact that a degree of arbitrariness attaches to the concept of maximum safe speed. The absolute maximum speed at which an individual driver is safe depends as much on the driver’s skill and reaction time, the quality and condition of the vehicle and its tyres, the weather conditions and the time of day (insofar as this affects visibility) as on the design features of the road.

Where it is necessary to vary the design speed along a section of road because of topographic or other limiting features, care should be taken to ensure that adequate transitions from higher to lower standards are provided.

Ceiling speed

In the urban situation, the need to vary the design speed because of physically constraining features is not likely to arise with any frequency. However, situations in which it is desirable to reduce operating speeds are common. Cases in point are areas where localised high concentrations of pedestrian traffic prevail. Examples include in the vicinity of schools (with particular reference to primary and nursery schools), old-age homes, modal transfer points and hospitals. Activity streets, where mixed usage may prevail, may require low operating speeds over substantial distances.

It would be extremely unwise to reduce the design speed in these areas, since a reduction in the design speed carries with it a reduction of sight distance. With the greater number of potential hazards that need to be observed and responded to, the driver should be afforded as much sight distance, and hence reaction time, as possible. In such areas, the design speed should be increased rather than reduced. An increase in the design speed by a factor less than 1,2 is not likely to produce any significant difference in operating conditions as perceived by the driver.

Clearly, however, the higher design speed should not serve as an inducement to increase operating speed and the concept of traffic calming would have to be brought into play.

Table 7.5 offers design and ceiling speeds appropriate to various classes of roads and streets. It should be noted that, ideally, shopping precincts such as malls should be so designed that vehicular access to them is not necessary. Should this not be possible to achieve in practice, access should be permitted only outside normal business hours. Parking areas serving shopping precincts should be designed to minimise vehicle-pedestrian conflicts.

Design hour

In the same way that a design speed is not a speed, the design hour is not an hour in the normal sense of the word. It is, in fact, a shorthand description of the conditions being designed for - specifically the projected traffic conditions.

These conditions include

- traffic volume, measured in vehicles per hour;
- traffic density, measured in vehicles per kilometre;

Table 7.5: Recommended design and ceiling speed

CLASS OF ROAD	DESIGN SPEED (km/h)	CEILING SPEED (km/h)
Vehicles only (freeways)	100 - 120	Not applicable
Vehicles only (other)	70 - 100	Not applicable
Mixed (higher order)	60 - 80	50 - 60
Mixed (middle order)	40 - 60	30 - 50
Mixed (lower order)	40 - 60	30 - 50
Pedestrian	30	20 - 30
Shopping precincts	30	<20

- traffic composition (i.e. the proportion of passenger cars, buses, rigid-chassis trucks and articulated vehicles comprising the traffic stream and usually expressed in percentage form); and

- directional split which, in an urban peak hour, readily achieves values of 80:20 or worse. Tidal flow implies that the 80:20 split in the evening peak would be in the opposite direction to that experienced in the morning peak *and both have to be designed for.*

The design hour is thus a combination of two distinctly different sets of circumstances, i.e. the morning and the afternoon peak in the case of commuter routes. Other routes may have different characteristics defining peak flows. Furthermore, the peak period may have a duration that is longer (or shorter) than 60 minutes and contain within itself a shorter period (typically 15 minutes) with very intense traffic flows.

A design life of 20 years is often assumed as a basis for design. This period may be altered subject to the planning of the authority concerned, and the evaluation of the economic consequences of departure from the suggested time span. For example, a road carrying low traffic volumes with few buses or trucks in the traffic stream may justify a shorter design life because of the savings accruing from the smaller number of axle-load repetitions in the shorter period. These savings arise from a reduction in the thickness of the design layers of the pavement and possibly even from a reduction in the quality of the materials required for road construction. A road carrying high volumes of bus or truck traffic in very hilly terrain may require a longer design life to achieve a reasonable return on the initial cost of construction.

Traffic volumes are usually expressed in terms of average daily traffic (ADT) measured in vehicles per day, with the ADT referring to an extended period, typically of the order of a year. Reference is made to Annual Average Daily Traffic (AADT) only if traffic counts are available for the period 1 January to 31 December. The ADT does not reflect monthly or daily fluctuations in traffic volume unless the month or day is explicitly specified.

The design hourly volume is frequently assumed to be the 30th highest hourly volume of the future year chosen for design, i.e. the hourly volume exceeded during only 29 hours of that year. The design hourly volume is expressed as a percentage of the ADT and typically varies from 12 to 18%. A value of 15% is thus normally assumed unless actual traffic counts suggest another percentage. Major urban links subject to commuter flows have a relatively low variation in flow when flows are ordered from highest to lowest across the number of hours in the year and, very often, the 100th highest hourly flow (at about 10 to 12% of the ADT) is an adequate basis for design.

Assessment of the total daily volume and hence hourly flows to be accommodated is a matter of some complexity. In the rural situation, naive modelling (i.e. applying a simple growth factor to present-day traffic counts) is adequate because changes in the nature and intensity of land use are slow if, in fact, they occur at all. In the urban situation, however, these changes are both significant and rapid. Furthermore, alternative routes are available. More sophisticated forms of modelling are necessary and application of naive modelling is not recommended.

The road network is really intended to support passenger trips or freight trips, with vehicle trips being almost incidental. Where would-be trip makers have a choice of mode, namely a convenient, safe, economical public transport service (which may be bus, rail bus or light rail) that really competes with the passenger car, traffic flows could be substantially lower than otherwise anticipated.

Density is a function of flow and speed, as illustrated by the units of measurement involved.

$$\text{Density (veh/km)} = \frac{\text{Flow (veh/h)}}{\text{Speed (km/h)}}$$

This function is not quite as direct as the equation implies because the basis of measurement differs. Density is measured across a considerable length of street at a single point in time, whereas flow is measured at a single point in space over an extended period of time. For this reason, the speed referred to in the above relationship is space mean speed as opposed to the more generally understood time mean speed.

It is the relationship between these measures which defines the Level of Service to be provided by the street being designed and, hence, the number of moving lanes to be provided in the cross-section.

SIGHT DISTANCE

Sight distance is a fundamental criterion in the design of any road or street. It is essential for the driver to be able to perceive hazards on the road, with sufficient time in hand to initiate any required action safely. On a two-lane two-way road it is also necessary for him or her to be able to enter the opposing lane safely while overtaking. In intersection design, the application of sight distance is slightly different from that applied in design for the rest of the road or street system but safety is always the chief consideration.

Stopping sight distance (SSD)

Stopping distance involves the ability of the driver to bring the vehicle safely to a standstill and is thus based on speed, driver reaction time and skid resistance. The total distance travelled in bringing the vehicle to a stop has two components:

- the distance covered during the driver's reaction period; and
- the distance required to decelerate to 0 km/h.

The stopping distance is expressed as :

$$s = 0{,}694\,v + v^2 / 254\,f$$

where: s = total distance travelled (m)
v = speed (km/h)
f = brake force coefficient

Stopping sight distances are based on operating speeds. The brake-force coefficients quoted in Table 7.4 have been adopted for design, and the calculated stopping sight distances are given in Table 7.6.

Stopping sight distance is measured from an eye height of 1,05 m to an object height of 0,15 m in the case of the higher-order roads. This object height is used because an obstacle of a lower height would not normally represent a significant hazard. In residential areas, the object height can be increased to 0,6 m. This greater height provides a practical design with an adequate margin of safety for the protection of children, pets and other obstacles typically encountered on this class of street.

Object height is also taken into account because, if the sight distance were measured to the road surface, the length of the vertical curve required would be substantially increased.

This could result in streets being significantly above or below natural ground level. In the urban environment where there is a need for access to adjacent properties at relatively short intervals, this is not acceptable.

The gradient has a marked effect on the stopping-distance requirements. Figure 7.2 is an expansion of Table 7.6, demonstrating this effect.

Table 7.6: Stopping sight distance on level roads

DESIGN SPEED (km/h)	STOPPING SIGHT DISTANCE (m)
30	30
40	50
50	65
60	80
70	95
80	115
90	135
100	155
110	180
120	210

Stopping sight distance can also be affected by a visual obstruction such as a garden wall or shrubbery next to the lane on the inside of a horizontal curve, as shown in Figure 7.3.

Barrier sight distance (BSD)

Barrier sight distance is the limit below which overtaking is legally prohibited. Two opposing vehicles travelling in the same lane should be able to come to a standstill before impact. A logical basis for the determination of the barrier sight distance is therefore that it should equal twice the stopping distance, plus a further distance of 10 m to allow an additional safety margin. The values given in Table 7.7 reflect this approach.

Barrier sight distance is measured to an object height of 1,3 m, with eye height remaining unaltered at 1,05 m. The greater object height is realistic because it represents the height of a low approaching vehicle.

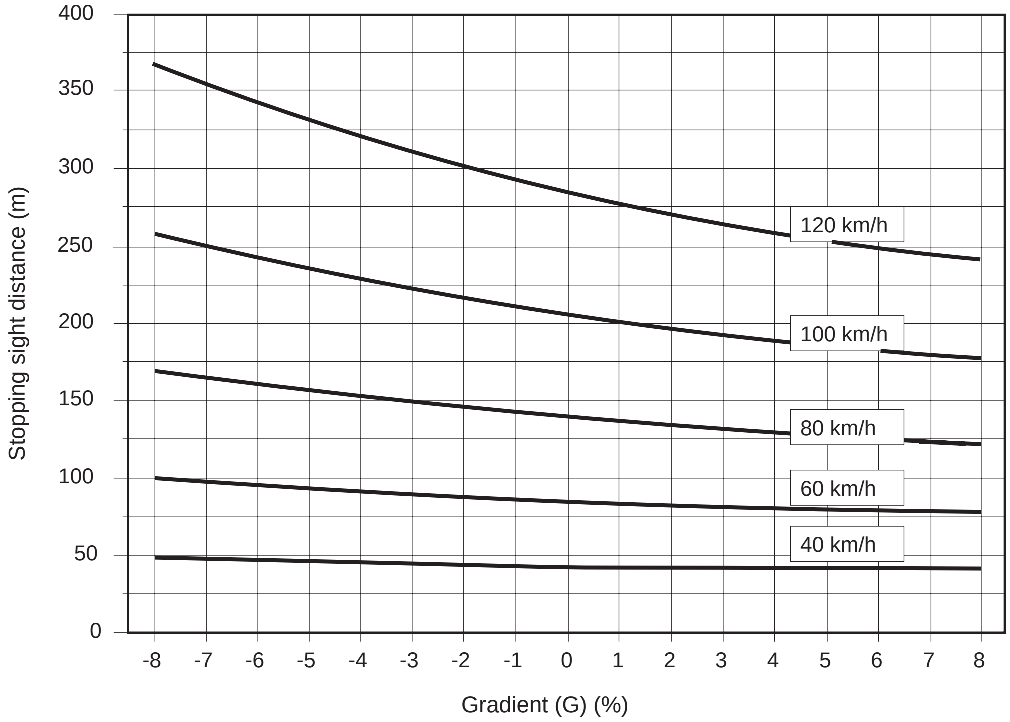


Figure 7.2: Stopping sight distance on gradients

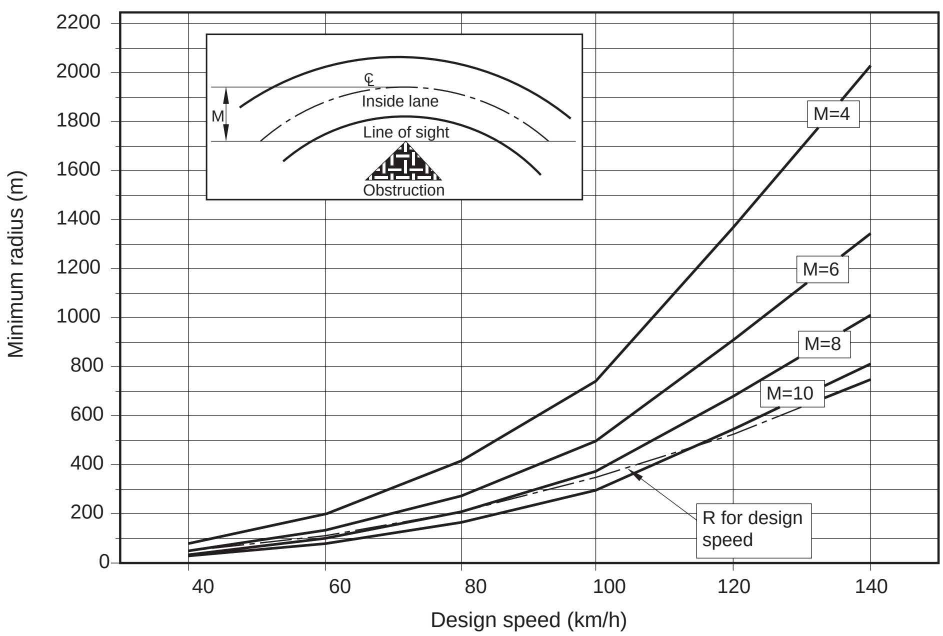


Figure 7.3: Minimum horizontal radius for stopping sight distance

Table 7.7: Barrier sight distance	
DESIGN SPEED(km/h)	BARRIER SIGHT DISTANCE(m)
40	110
60	170
80	240
100	320
120	430

Hidden dip alignments are commonly considered to be poor design practice. They typically mislead drivers into believing that there is more sight distance available than actually exists. In checking the alignment in terms of barrier sight distance, the designer should pay detailed attention to areas where this form of alignment occurs, to ensure that drivers are made aware of any inadequacies of design.

Because of the low speeds involved and the typically short lengths of lower-order mixed-usage streets, the passing operation is of little significance so that barrier markings are seldom, if ever, employed on these streets.

Decision sight distance (DSD)

The best visual cue to the driver is the roadway ahead. For this reason it is necessary in certain circumstances for the road surface itself to be visible to the driver for a given distance ahead. This is to allow sufficient time for the assimilation of a message and the safe initiation of any action required. An example is the marking that allocates specific lanes at an intersection to turning movements. Warning of this must be given sufficiently far in advance of the intersection to permit a lane change that does not detrimentally affect the operation of the intersection itself.

Decision sight distance, as given in Table 7.8, is related to the reaction time involved in a complex driving task. The reaction time selected for this purpose is 7,5 seconds, which is roughly the mean of values quoted in American practice. The calculated values in Table 7.8 are based on stopping sight distance to allow for the condition where the decision is to bring the vehicle to rest.

Table 7.8: Decision sight distance on level roads

DESIGN SPEED (km/h)	STOPPING SIGHT DISTANCE (m)
40	130
60	190
80	240
100	300
120	350

This has the effect of increasing the normal reaction time of 2,5 seconds by a further five seconds of travel at the design speed of the road. Decision sight distance is measured from an eye height of 1,05 m to the road surface, i.e. to an object height of 0 m.

Passing sight distance (PSD)

In the case of vehicles-only and higher-order mixed usage streets, passing sight distance is an important criterion indicative of the quality of service provided by the road. The initial design is required to provide stopping sight distance over the full length of the road, with passing sight distance being checked thereafter. A heavily trafficked road requires a higher proportion of passing sight distance than a lightly trafficked road to provide the same level of service. Insufficient passing sight distance over a vertical curve can be remedied, for example, either by lengthening the vertical curve to provide passing sight distance within the length of the curve itself, or by shortening the curve to extend the passing opportunities on either side. Horizontal curves can similarly be lengthened or shortened. A further possibility is the provision of a passing lane.

Passing sight distance can be calculated on one of two bases, being either the sight distance required for a successful overtaking manoeuvre or that required for an aborted manoeuvre. The former could be described as being a desirable standard and the latter as the minimum. Values quoted for the successful manoeuvre are taken from AASHTO (1994) and for the aborted manoeuvre from Harwood and Glennon (1989), who base these distances on the vehicles involved being a passenger car passing a bus or a truck.

Table 7.9 lists passing sight distances in respect of both successful and aborted manoeuvres.

Table 7.9: Passing sight distance on level roads

DESIGN SPEED (km/h)	PASSING SIGHT DISTANCE (m)	
	SUCCESSFUL MANOEUVRE	ABORTED MANOEUVRE
40	290	-
60	410	226
80	540	312
100	670	395
120	800	471

Passing sight distance in respect of a successful manoeuvre allows adequately (according to Harwood and Glennon 1989) for an aborted manoeuvre in the case of a bus or truck attempting to pass another.

As in the case of barrier sight distance, passing sight distance is not a consideration in the design of lower-order mixed-usage streets.

Intersection sight distance (ISD)

At a stop-controlled intersection, the driver of a stationary vehicle must be able to see enough of the through-road or street to be able to carry out one of three operations before an approaching vehicle reaches the intersection, even if this vehicle comes into view just as the stopped vehicle starts to move. These three operations are to:

- turn to the left in advance of a vehicle approaching from the right;
- turn to the right, crossing the path of a vehicle approaching from the right and in advance of a vehicle approaching from the left;
- to move across the major highway in advance of a vehicle approaching from the left.

In the first case, the assumption is that the turning vehicle will accelerate to 85% of the design speed of the through-road and a vehicle approaching on the through-road will decelerate from the design speed also to 85% of the design speed, leaving a two-second headway between them at the end of the manoeuvre.

According to AASHTO, the intersection sight distance required for the right turn is only about one metre less than that required for the left turn, given the same assumptions as made in the first case.

In the case of the vehicle crossing the through- road, the distance the crossing vehicle must travel is the sum of:

- the distance from the stop line to the edge of the through carriageway;
- the width of the road being crossed; and
- the length of the crossing vehicle.

This manoeuvre must be completed in the time it takes the approaching vehicle to reach the intersection, assuming that the approaching vehicle is travelling at the design speed of the through-road. For safety, the time available should also include allowance for the time it takes for the crossing driver to establish that it is safe to cross, engage gear and set his or her vehicle in motion: a period of about two seconds is normally used.

Intersection sight distances recommended in accordance with the principles outlined above are given in Figures 7.4 and 7.5. Before a lower value is adopted in a specific case, the implications of departing from the recommended values should be considered.

The line of sight is taken from a point on the centre line of the crossing road and 2,4 m back from the edge of the through-road, to a point on the centre line of the through-road, as shown in Figures 7.4 and 7.5. The setback is intended to allow for a pedestrian or cycle track crossing beyond the Stop line.

The object height is 1,3 m. The eye height is 1,05 m for a passenger car and 1,8 m for buses and all other design vehicles. There should not be any obstruction to the view in the sight triangle, which is defined as the area enclosed by the sight line and the centre lines of the intersecting roads.

Where an intersection is subject to yield control, the unobstructed sight triangle must be larger. If it is assumed that the vehicle approaching the intersection on the minor leg will be travelling at 30 km/h, a distance of 30 m would be required to stop the vehicle. If the driver is already preparing to stop, allowance for reaction time is no longer necessary and a distance of 10 m is required to bring the vehicle to a standstill. If the approach speed is 60 km/h, the required distance is 45 m.

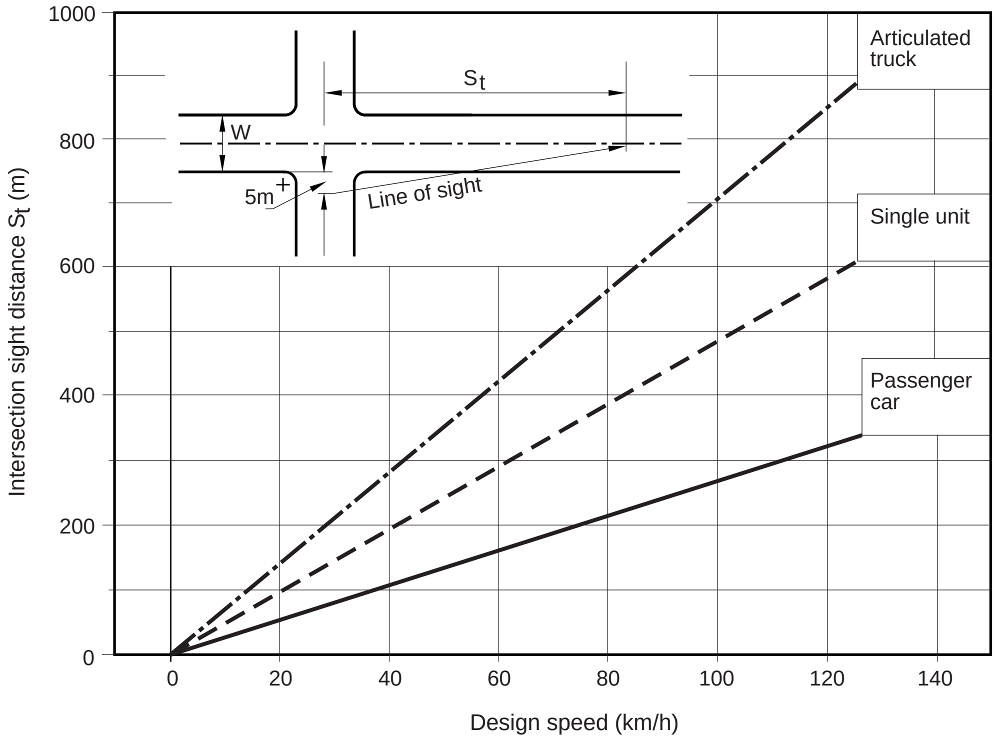


Figure 7.4: Intersection sight distance for turning manoeuvre

The sight triangles required for yield control based on an approach speed of 60 km/h are so large that the probability of their being found in an urban area is remote.

If the driver does not stop but turns to travel in the same direction as a vehicle approaching at the design speed of the through-road, the driver of the latter vehicle will be forced to slow down to match speeds at a safe following distance.

The intersection sight distance for this manoeuvre is shown in Figure 7.6

Because the driver approaching the yield sign may be required to stop, intersection sight distance as defined and measured for the stop condition must also be available.

Intersections are, typically, the points at which pedestrians would want to cross the through-road or

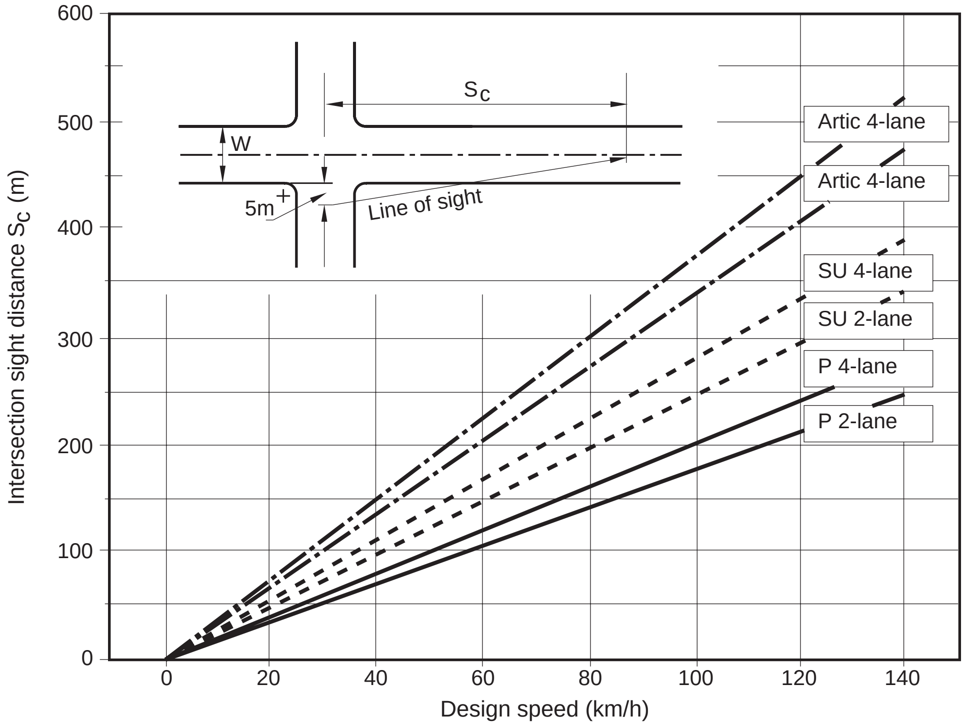


Figure 7.5: Intersection sight distance for crossing manoeuvre

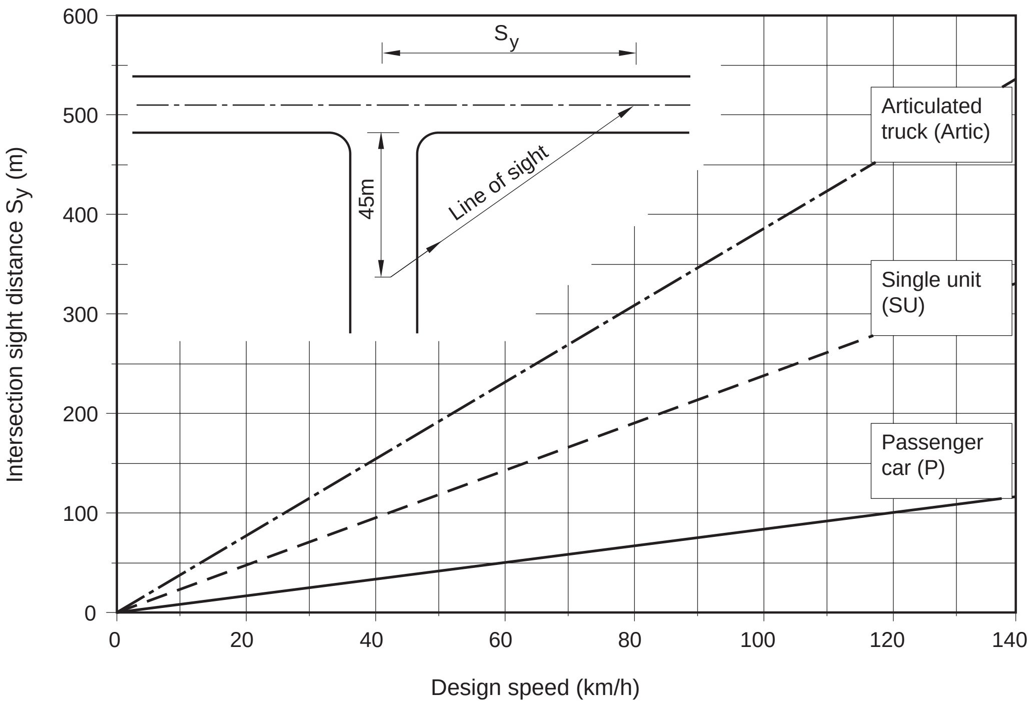


Figure 7.6: Intersection sight distance for yield condition

street. Pedestrians must therefore be provided with adequate sight distance to ensure that they can cross the through-street in safety. This case is precisely analogous to that of the vehicle at the intersection, because the principle involved is that the sight distance provided is directed towards what the pedestrian must be able to see rather than the sight distance available to drivers of vehicles on the through-road.

Pedestrian sight distance is measured from an eye height of 1,0 m to an object height of 1,3 m. It is assumed that the pedestrian is located on the left side of the intersecting street with the oncoming vehicle approaching also from the left. This represents the longest crossing distance before a situation which is at all safe is achieved, because the further assumption is that the pedestrian will not be required to pause on the centreline of the through-road. The distances offered in Table 7.10 would be adequate for crossing a two-lane road.

Table 7.10: Pedestrian sight distance

DESIGN SPEED (km/h)	SIGHT DISTANCE(m)
30	45
40	55
50	70
60	85
70	100

If adequate sight distance is not available, it may be necessary to provide a signalised cross-walk, thus forcing through vehicles to stop. Furthermore, if an adequate gap in the through-traffic does not present itself at intervals not exceeding one minute, a signalised pedestrian crossing should also be considered.

HORIZONTAL ALIGNMENT

The horizontal alignment of a road or street is the combination of curves and straights (or tangents) presented on a plan view. Curves are usually circular, although spirals and other higher-order polynomials can be used under highly specific circumstances, which are seldom found in residential environments.

Determination of the horizontal alignment of an urban street is a planning rather than a detailed design function, and is highly iterative in nature. Iteration is not only between the three dimensions of design, e.g. where restraints in the vertical dimension may force a shift in horizontal alignment, but also involves continuous revisiting of the intentions originally formulated with regard to settlement making.

Design of the horizontal alignment must also give effect to the proposed function of the road or street. For example, the horizontal alignment of a freeway is typified by long tangents and gentle curves, whereas a residential street should be designed to discourage operating speeds higher than 40 to 50 km/h.

General principles to be observed in the determination of the horizontal alignment of a road or street are the following:

- No vehicle can instantaneously change from traversing a curve in one direction to traversing one in the reverse direction. Short lengths of tangent should thus be used between reverse curves.

- Broken-back curves (where two curves in the same direction are separated by a short tangent) should not be used as they are contrary to drivers' expectations. In the residential environment, this is difficult to avoid as cadastral boundaries are straight lines. Fitting smooth curves within a reserve comprising a series of chords of a circle is not always possible.

- Large- and small-radius curves should not be mixed. Successive curves to the left and the right should generally have similar radii and the 1:1,5 rule is a useful guide in their selection.

- In residential areas, the deviation angle of short-radius curves should not exceed 90° as, at higher values of deviation, encroachment by large vehicles on opposing lanes becomes pronounced and, furthermore, the splay that has to be provided to permit adequate sight distance becomes excessive.

- For small-deflection angles, curves should be sufficiently long to avoid the impression of a kink.

- Alignment should be sensitive to the topography to minimise the need for cuts and fills and the restriction that these place on access to erven from the street. Streets at right angles to the contours can create problems in terms of construction, maintenance, drainage, scour (in the case of gravelled surfaces) and also constitute a traffic hazard. During heavy rainstorms, water flowing down a steep street can flow across the intersecting street.

In addition, the various utilities, such as sewerage, power and water reticulation, are typically located within the road reserve. The planning of the road network must therefore also take cognisance of the limitations to which these services are subject. For example, a street located in such a fashion that its vertical alignment tends to be undulating would present significant difficulties in the location of sewer runs.

Tangents

As horizontal curves are circular, the straights connecting them are usually referred to as tangents. While the selection of radius of horizontal curvature dictates the operating speed selected by the driver, long tangents can cause speeds to increase to unacceptable levels, followed by deceleration as the next curve is encountered. It has been found that limiting the length of tangents (in metres) to about ten times the design speed (in km/h) will cause speeds to stay fairly constant. A design speed of 40 km/h would thus suggest that tangents should not be more than about 400 m in length.

In the situation of an urban grid of streets, a 400-metre-long tangent bounded at both ends by T-intersections would also tend to limit speeds to of the order of 40 km/h.

Curvature and superelevation

Acceptable rates of superelevation are offered in Table 7.11.

Table 7.11: Maximum superelevation for various classes of road

CLASS OF ROAD OR STREET	MAXIMUM SUPERELEVATION (%)
Vehicle-only (freeway)	10
Vehicle-only (other)	6 - 8
Mixed-usage (higher-order)	4 - 6
Mixed-usage (middle-order)	2 - 4
Mixed-usage (lower-order)	2 - 4

In Table 7.12, the minimum radii of horizontal curves for various design speeds and maximum rates of superelevation are calculated from the relationship

$$R = \frac{v^2}{127(e + f)}$$

where: R = radius (m)
v = speed (km/h)
e = superelevation rate(m/m)
f = side friction factor

Unlike the rural situation where a tight radius curve can be matched by a high value of superelevation, the large variations in vehicle speeds encountered in the urban environment cause high values of superelevation to be inappropriate. Furthermore, there is a distinct likelihood that there would not be sufficient distance available to accommodate the development of superelevation. Property access in the immediate vicinity of the curve probably would not allow a cross-section where one road edge is a metre or more above the other.

In Table 7.12, all values have been rounded up to the nearest five metres. A camber of 2 to 3% suggests that rates of superelevation of -0,02 to +0,02 are usually normal camber situations. There is no known application for a 0% super-elevation and it should be avoided as, in the absence of a longitudinal gradient, it will cause drainage problems and ponding on the road surface.

Superelevation runoff

Streets normally have a camber with the high point on their centreline and a fall, typically of the order of 2 to 3% as suggested above, to either edge. Superelevation is developed or run off by rotating the outer lane around the centreline until a crossfall across the full width of the street, equal to the original camber, is achieved. From this point, both lanes are further rotated around the centreline until the full extent of superelevation has been achieved.

This further rotation need not necessarily be about the centreline. Special circumstances may demand a different point of rotation. A constraint on the level of one or other of the road edges may require that the constrained edge becomes the axis of rotation. The need to secure an adequate, but not too steep, fall to a drop inlet on the inside of a curve may require that the axis of rotation be shifted to a point slightly

Table 7.12: Minimum radii for horizontal curves (m)

DESIGN SPEED (km/h)	SIDE FRICTION FACTOR (f)	MINIMUM RADII FOR MAXIMUM RATES OF SUPERELEVATION (e) OF:					
		-0,02	0	+0,02	0,04	+0,06	+0,08
30	0,19	45	40	35	30	30	30
40	0,18	80	70	65	60	55	50
50	0,17	135	115	105	95	85	80
60	0,16	205	180	160	145	130	120
70	0,15	300	260	230	205	185	170
80	0,14	420	360	315	280	255	230
100	0,13	-	-	525	465	415	375
120	0,11	-	-	875	760	670	600

removed from the inner edge.

Rotation over too short a distance will create the impression of an unsightly kink in the road surface and, if the distance is too long, drainage problems are likely to occur in the area where the camber is less than about 0,5%. The rate of rotation is measured by the relative slope between the roadway edge and the axis of rotation. Relative slopes that have been found in practice to give acceptable lengths of runoff are quoted in Table 7.13. Where space does not permit the use of these rates, minimum lengths for superelevation runoff for two-lane roads may have to be adopted. These are also quoted in Table 7.13. These lengths are based on relative slopes that are generally 50% higher than those recommended for normal use.

Table 7.13: Rates and minimum lengths of superelevation runoff

DESIGN SPEED	RELATIVE SLOPE (%)	MINIMUM LENGTH (m)
40	0,7	35
60	0,6	40
80	0,5	50
100	0,4	60
120	0,4	70

Where a circular arc is preceded by a transition curve, the full superelevation is developed across the length of the transition. Transition curves, however, are only used on the tightest radius curves applied to roads with high design speeds. In all other cases, the superelevation runoff must be distributed between the tangent and the curve because full superelevation at the end of the tangent is as undesirable as no superelevation at the start of the curve. Drivers tend to follow a transition path in entering a curve and this path typically has two-thirds of its length on the tangent with the remaining third being on the curve itself. Superelevation runoff is similarly distributed to match the actual path of the vehicle.

VERTICAL ALIGNMENT

Vertical alignment is the combination of parabolic vertical curves and straight sections joining them. Straight sections are referred to as grades, and the value of their slope is the gradient, usually expressed in percentage form, e.g. a 5% grade climbs through 5 metres over a horizontal distance of 100 metres.

With the whole-life economy of the road in mind, vertical alignment should always be designed to as high a standard as is consistent with the topography. Passenger car speeds are dictated by the standard of horizontal alignment rather than by the vertical alignment, whereas the speeds of buses and other heavy vehicles are constrained more by the vertical alignment. The design speed applied to the vertical alignment should therefore match that applied to the horizontal alignment and it could be argued that a higher vertical design speed is preferable.

As in the case of the horizontal alignment, the vertical alignment should be designed to be aesthetically pleasing. In this regard due recognition should also be given to the interrelationship between horizontal and vertical curvature. A vertical curve that coincides with a horizontal curve should, if possible, be contained within the horizontal curve and, ideally, have approximately the same length.

Where a vertical curve falls within a horizontal curve, the superelevation generated by the horizontal curvature improves the availability of sight distance beyond that suggested by the value of vertical curvature. This enables the edge profiles to have a curvature sharper than the minima suggested in Table 7.14. The proviso, however, is that the driver's line of sight is contained within the width of the roadway. When the line of sight goes beyond the roadway edge, the effect on sight distance of lateral obstructions such as boundary walls or high vegetation must be checked.

A smooth grade line with gradual changes appropriate to the class of road and the character of the topography is preferable to an alignment with numerous short lengths of grade and vertical curves. The “roller coaster” or “hidden dip” type of profile should be avoided. This profile is particularly misleading in terms of availability of sight distance and, where it cannot be avoided, sight distance greater than suggested in Table 7.6 may be required in terms of accident experience. For aesthetic reasons, a broken-back alignment is not desirable in sags where a full view of the profile is possible. On crests the broken-back curve adversely affects passing opportunity.

Curvature

The horizontal circular curve provides a constant rate of change of bearing. Analogous to this is the vertical parabola which provides a constant rate of change of gradient. Academic niceties apart, there is little to choose between the application of the parabola or the circular curve, the differences between them being virtually unplottable and, in any event, within the levels of accuracy to which the pavement typically is constructed.

From the general form of a parabolic function

$$y = ax^2 + bx + c$$

it follows that the rate of change of grade, d^2y/dx^2, equals 2a. The reciprocal of 2a, K, is thus the distance required to effect a unit change of grade. Vertical curves are specified in terms of this factor, K, and their horizontal length as shown in the relationship

$$L = A.K$$

Minimum rates of curvature

The minimum rate of curvature is determined by sight distance as well as by considerations of comfort of operation and aesthetics. The sight distance most frequently employed is the stopping sight distance which, as stated earlier, is measured from an eye height of 1,05 m to an object height of 0,15 m although, in the case of residential streets, an object height of 0,6 m could be used.

In the case of sag curves, the sight distance is replaced by a headlight illumination distance of the same magnitude, assuming a headlight height of 0,6 m and a divergence angle of 1° above the longitudinal axis of the headlights. Where adequate street lighting is available, the headlight criterion does not apply and comfort is the only criterion that limits values.

Special circumstances may dictate the use of decision sight distance or even passing sight distance. Where a sight distance other than that for stopping has to be employed, the relationship offered below can be used to calculate the required curve length and, thereafter, the K-value of vertical curvature.

Where the sight distance, S, is less than the curve length, L,

$$L = \frac{AS^2}{100\,(\sqrt{2h_1} + \sqrt{2h_2})}$$

and, where S is greater than L,

$$L = 2S - \frac{200\,(\sqrt{h_1} + \sqrt{h_2})^2}{A}$$

where: L = length of vertical curve (m)
S = sight distance (m)
A = algebraic difference in grades (%)
h_1 = height of eye above road surface (m)
h_2 = height of object above road surface (m)

Values of K, based on stopping sight distance in the case of crest curves, and on headlight illumination distance in the case of sag curves, are given in Table 7.14.

Minimum lengths of vertical curves

Where the algebraic difference between successive grades is small, the intervening minimum vertical curve becomes very short, and, particularly where the adjacent tangents are long, the impression of a kink in the grade line is created. Where the difference in grade is less than 0,5%, the vertical curve is often omitted. In Table 7.15, a minimum length of curve for algebraic differences in grade greater than 0,5% is suggested for purely aesthetic reasons.

Where a crest curve and a succeeding sag curve have a terminal point in common, the visual effect created is that the road has suddenly dropped away. In the reverse case, the illusion of a hump is created. Either effect is removed by inserting a

Table 7.14: Minimum values of K for vertical curves

DESIGN SPEED (km/h)	CREST CURVES FOR OBJECT OF HEIGHT		SAG CURVES	
	0,15 m	0,60 m	Without street lighting	With street lighting
40	6	2	8	4
50	11	6	12	6
60	16	10	16	8
70	23	Not applicable	20	Not applicable
80	33		25	
90	46		31	
100	60		36	
110	81		43	
120	110		52	

short length of straight grade between the two curves and, typically, 60 m to 100 m is adequate for this purpose.

Table 7.15: Minimum length of vertical curves

DESIGN SPEED (km/h)	LENGTH OF CURVE (m)
40	80
60	100
80	140
100	180
120	220

Gradients

Maximum gradients on higher order roads

Bus and truck speeds are markedly affected by gradient. Bus routes should be designed with gradients which will not reduce the speed of these vehicles enough to cause intolerable conditions for following drivers. Glennon (1970) found that the frequency of accidents increases sharply when the speeds of heavy vehicles are reduced by more than 15 km/h.

For southern African conditions a speed reduction of 20 km/h is generally accepted as representing intolerable conditions. If gradients on which bus or truck speed reduction is less than 20 km/h cannot be achieved economically, it may be necessary to provide auxiliary lanes for the slower-moving vehicles. Wolhuter (1990) established that, on flat grades, 50 percentile bus and truck speeds are about 17 km/h lower than the equivalent passenger car speeds, so that a speed reduction of 20 km/h actually represents a total speed differential of about 37 km/h.

Maximum gradients for different design speeds and types of topography are suggested in Table 7.16. It is stressed that these are guidelines only. Optimisation of the design of a specific road, taking the whole-life economy of the road into account, may suggest some other maximum gradient.

The three terrain types described are defined by the differences between passenger-car and bus or truck speeds prevailing in them. On flat terrain, the differences between the speeds of cars and buses remain relatively constant at about 17 km/h, whereas hilly terrain causes substantial speed differentials. In mountainous terrain, buses and trucks are reduced to crawl speeds for substantial distances.

Maximum gradients on residential streets

On local streets, maximum gradient has a significant effect on the cost of township development. Where possible, road alignment should be designed to minimise the extent and cost of earthworks and to avoid problems with access and house design. It therefore has to be accepted that short sections of steep gradients may be necessary in some settlement developments.

Where a residential street is also a bus route, the gradients recommended in Table 7.16 should not be exceeded. Where this is not possible, a maximum gradient of 14% may have to be considered.

On higher-order mixed-usage streets, the recommended maximum gradient is 10% but, on sections not longer than 70 m, the gradient can be increased to 12,5%. On purely residential streets, the maximum gradient could be 12% and on sections not longer than 50 m the gradient could be increased to 16%.

Notwithstanding the values given, the following points should be taken into consideration:

- Gradients should be selected in consultation with the stormwater design engineer.
- Steep gradients on short access loops and culs-de-sac could result in properties being inundated and surface runoff washing across

Table 7.16: Maximum gradients on major roads (%)

DESIGN SPEED (km/h)	TOPOGRAPHY		
	FLAT	ROLLING	MOUNTAINOUS
40	7	8	9
60	6	7	8
80	5	6	7
100	4	5	6
120	3	4	5

intersecting streets.

- Multiple-use surfaces which serve both vehicular access and recreational purposes, including playing space for children, should be relatively flat and not provided with kerbs (they are, in fact, shared surfaces).

- Where cycling is an important mode of travel, it will be necessary to consider the effects of gradient on cycling in deciding on the road alignment.

- It is difficult to construct streets on gradients steeper than about 12% by conventional means. 12/14 ton rollers cannot climb gradients this steep. They also tend to damage the base course while attempting to stop after a downhill pass. Steeper grades should thus be constructed of concrete, brick or interlocking road stones. The last-mentioned surface is not recommended where speeds in excess of about 60 km/h are anticipated, as the partial vacuum created behind a passing tyre tends to suck out the sand from between adjacent stones and thus destroy the integrity of the surface.

- Gravel surfaces are subject to scour at water flow speeds of the order of 0,6 to 1,0 m/s. Under conditions of overland flow, this speed is achieved at slopes of the order of 7 to 8%. The slope in question is the resultant of the vectors of longitudinal slope and crossfall.

Minimum gradients

If the cross-section of the road does not include kerbing, the gradient could be 0% because the camber is continued across the adjacent shoulder, thus allowing for adequate drainage of the road surface. The verge will have to accommodate the drainage both of the road reserve and of the surrounding properties. The decision to accept a zero gradient would thus have to be informed by the stormwater drainage design. Zero gradient is not recommended as a general rule and the preferred minimum is 0,5%.

Kerbed streets should have a minimum gradient of not less than 0,5%. If the street gradient has to be less than this, it would be necessary to grade the kerbs and channels separately and to reduce the spacing between drop inlets to ensure that the height difference between the edge of the travelled way and channel is not too pronounced.

Climbing lanes

Application of climbing lanes

Climbing lanes are auxiliary lanes added outside the through-lanes. They have the effect of reducing congestion in the through-lanes by removing slower-moving vehicles from the traffic stream. As such, they are used to match the Level of Service on the rising grade to that prevailing on the level sections of the route. In the urban situation, climbing lanes may be used on vehicles-only and higher-order mixed-usage streets. They have no application on local residential streets.

Warrants for climbing lanes

As implied earlier, the maintenance of an acceptable level of service over a section of the route is one of the reasons for the provision of climbing lanes. Another reason is the enhancement of road safety by the reduction of the speed differential in the through-lane. The warrants for climbing lanes are therefore based on both speed and traffic volume.

A bus/truck speed profile should be prepared for each direction of flow. It would then be possible to identify those sections of the road where speed reductions of 20 km/h or more may warrant the provision of climbing lanes.

The traffic volume warrant is given in Table 7.17. It should be noted that the word “trucks” includes buses, rigid-chassis trucks and articulated vehicles.

A further warrant is based on matching Levels of Service (LOS) along the route. Alternatively, a form of partial economic analysis developed by Wolhuter (1990) could be used. This software - ANDOG (**AN**alysis of **D**elay **O**n **G**rades) - is available from CSIR-Transportek. It compares the cost of construction of the climbing lane to the costs of the delay incurred by not providing it.

Location of terminals

A slow-moving vehicle should be completely clear of the through-lane by the time its speed has dropped by 20 km/h, and remain clear of the through-lane until it has accelerated again to a speed which is 20 km/h less than its normal speed. The recommended taper length is 100 m so that the start taper begins 100 m in advance of the point where the full climbing lane width is required, and the end taper ends 100 m beyond the end of the climbing lane.

If there is a barrier line, owing to restricted sight distance, at the point where the speed reduction warrant falls away, the full lane should be

Table 7.17: Traffic volume warrants for climbing lanes

GRADIENT (%)	TRAFFIC VOLUME IN DESIGN HOUR (veh/h)	
	5% trucks in stream	10% trucks in stream
4	632	486
6	468	316
8	383	257
10	324	198

extended to where the marking ends, with the taper ending 100 m beyond this point.

Climbing lane width

The climbing lane should preferably have the same width as the adjacent through-lanes. On major routes, through-lanes may have widths of 3,7 m, 3,4 m or 3,1 m. It is unlikely that climbing lanes will be provided on roads where the traffic volumes are so low that a lane width of 3,1 m is adequate. Climbing lanes therefore tend to be either 3,7 m or 3,4 m wide. Even if the through-lanes are 3,7 m wide, a climbing lane 3,4 m or perhaps even 3,1 m wide may, however, be considered on the grounds of low lane occupancy and speed or some other constraining topographic circumstance. Climbing lanes on bus routes should, however, have a width of 3,7 m.

CROSS-SECTION DESIGN

The cross-section of a road provides accommodation for moving and parked vehicles, drainage, public utilities, non-motorised vehicles and pedestrians. It is also required to serve more than just movement-related activities.

Residential streets, for example, offer a neutral territory on which neighbours can meet informally. They can also serve as playgrounds for children in developments where plot sizes are too small for this purpose.

Abutting trading or light industrial activities in activity corridors may require sidewalks wider than those required purely for moving pedestrians. Pedestrians also “park”, in the sense of browsing through goods on offer (either in shop windows or by roadside vendors) or relaxing in a sidewalk café. In short, the road reserve is required to address a wide spectrum of activities. For this reason it was suggested previously that reference should be to “hard open space” with only a portion of this comprising the road reserve as previously understood.

Movement, as an activity served by the cross- section, comprises a spectrum of needs. One end of the spectrum of the movement function relates to pure mobility, as typified by the freeway and urban arterial. Vehicle movement is the sole concern and pedestrians are totally excluded from these roads. The other end of the spectrum is concerned with accessibility and the needs of the pedestrian. Vehicular movement may be necessary on these roads but it is tolerated rather than encouraged and is subject to significant restrictions. Between these two extremes, mixed usage is found with vehicular and non-vehicular activities sharing the available space. If these uses have to compete for their share of space, it can reasonably be stated that the design has failed to meet its objective.

The flexibility of the road reserve in accommodating such widely disparate needs derives from the disaggregated nature of the cross-section, as illustrated in Figure 7.7.

The cross-section may comprise all or some of the following components:

- Lanes
  - Basic
  - High Occupancy Vehicle (HOV) lanes
  - Auxiliary (turning or climbing)
  - Parking
  - Cycle

- Medians
  - Shoulders
  - Central island

- Shoulders
  - Verges
  - Sidewalks.

Lanes

Basic or through-lanes

Undivided roads may have either one lane in each direction (two-lane two-way roads) or more than one lane in each direction (multilane roads). Dual carriageway roads have two or more lanes in each direction separated by a median. Customarily, there is symmetry of through-lanes, and asymmetry

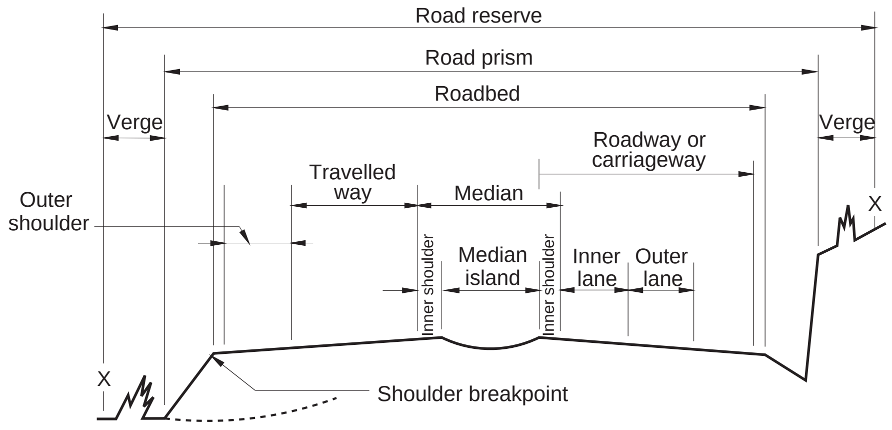


Figure 7.7: Elements of the cross-section

on a particular section of road should arise only from the addition of an auxiliary lane that is clearly allocated to one direction of travel.

The selection of lane width is based on traffic volume and vehicle type and speed. Higher volumes and speeds require wider lanes, and the greatest lane width recommended is 3,7 m. Where traffic volumes are such that a multilane cross-section or a divided cross-section is required, 3,7 m is a logical lane width to adopt.

No operational or safety benefit accrues from lane widths wider than 3,7 m, although some urban authorities allow lane widths as broad as 5,5 m. In peak hours, these wider lanes tend to carry two lanes of moving passenger cars each. They also ease the process of passenger cars overtaking buses without encroaching significantly on the opposing lane. Finally, they enable informal parking in the absence of demarcated parking bays. As such, 5,5 m lanes tend to be used only in higher-order mixed-usage streets.

The narrowest lane width recommended is 3,1 m, which gives a clear space of 0,25 m on either side of a vehicle that is 2,6 m wide i.e. a bus. This width would normally be employed only where speeds or traffic volumes are expected to be low and buses infrequent, e.g. on residential streets.

If the route is not intended ever to accommodate buses, the lane width could be reduced to as little as 2,7 m. Intermediate conditions of volume and speed can be adequately catered for by a lane width of 3,4 m.

Streets where pedestrian activities are expected to predominate may have only one lane, with provision for passing made at intervals. In this case, the lane width should not be less than 3,1 m.

Passing bays should be provided at not more than 50 m spacings. It is important that passing bays should be intervisible. If this is not achieved, motorists may enter a single lane section only to find that it is already occupied thus forcing one or other of the vehicles involved to reverse to the previous passing bay.

High occupancy vehicle (HOV) lanes

HOV lanes normally extend over considerable distances and could, therefore, be included in the category of basic lanes. These lanes are normally applied only to higher-order mixed-usage streets and are intended to serve all HOVs and not only buses. Vehicles that could be allowed to use HOV lanes thus include

- buses;
- minibus taxis; and
- car pool vehicles.

A policy decision would have to be provided by the local authority concerned in respect of the level of vehicle occupancy that would allow a vehicle to enter an HOV lane. An operational problem that immediately arises in the application of HOV lanes is their policing, to ensure that only vehicles legitimately described as HOVs use them.

As buses have an overall width of 2,6 m, the smaller basic lane widths would not be appropriate. As pointed out, a 3,1 m lane would allow only 0,25 m between the outside of the bus and the lane edge with a distinct possibility that a moving bus would not always be precisely located in the centre of the lane. At speeds higher than those encountered in residential areas, encroachment on other lanes could be expected. To avoid encroachment, a lane width of 3,7 m is the minimum that should be accepted for a HOV lane.

It is not possible to lay down hard-and-fast warrants for the provision of bus lanes. From an operational point of view, relating purely to the movement of people, it follows that the capacity of the street being analysed should be greater with the added HOV lane than without it. Where an existing lane is converted to an HOV lane, i.e. when it is no longer available for use by other vehicles, the provision of the HOV lane could lead to a decline in throughput of passengers.

The estimation of transit capacity is more complex and less precise than estimates of highway capacity as it deals with the movement of both people and vehicles whereas highway capacity restricts itself to vehicular movement. Furthermore, the variables to address in such estimates include, in addition to the normal factors applying to highway capacity

- the size of the transit vehicles, in terms of allowable passenger loadings;
- the frequency of operation of the service; and
- the interaction between passenger traffic concentrations and vehicle flow.

Reference should be made to TRB Special Report 209: Highway Capacity Manual with regard to the analysis of mass transit facilities.

The layout of bus stops is discussed under the heading “Verges”.

Auxiliary lanes

Auxiliary lanes are lanes added to the normal cross-section to address a specific purpose and are normally applied only to vehicle-only or higher-order mixed-usage streets.

Typically, auxiliary lanes are added at intersections to support left and right turns so that these manoeuvres can take place at relatively low speeds without impeding the movement of the through-traffic. If a road has signalised intersections, it may be necessary to add auxiliary lanes to match the intersection capacity to that of the approach legs. These lanes are discussed in more detail below under the heading “Intersections”.

Auxiliary lanes can also be provided at intersections to serve through-traffic. The intention is to match the capacity of the intersection to its upstream and downstream links. The need for such lanes and the storage length upstream and merging length downstream that they have to accommodate are a matter for analysis as described by the Highway Capacity Manual.

Climbing lanes, as auxiliary through-lanes, are discussed above under the heading “Vertical Alignment”.

Parking lanes

Parking lanes are normally 2,5 m wide with a minimum width of 2,1 m, and are usually embayed. In this configuration, the parking lane is actually located in the verge area. Individual parking bays are typically 6,0 m long with each pair of parking bays being provided with an additional clear space of 1,5 m between them to allow for manoeuvring into or out of the bays. In areas where very high tidal flows are expected, it is useful to be able to use the parking lanes as moving lanes during periods of peak flow. Under these circumstances, the parking lane should have a minimum width of 3,1 m.

Cycle lanes

Ideally, cycle lanes should be located in the verge area, as the speed differential between bicycles and pedestrians is likely to be less than that between bicycles and motorised vehicles. Where this is not possible and either there is significant cycle traffic or it is desired to encourage bicycles as a mode of travel, a cycle lane can be added outside those intended for motorised vehicles.

Such lanes should be of the order of 1,5 m wide and clearly demarcated as cycle lanes. If these lanes are wider than 2,0 m, passenger cars are likely to use them, possibly even for overtaking on the left, which is a manoeuvre to be actively discouraged.

Shoulders

The shoulder is defined as the usable area alongside the travelled way.

Shoulders are applied only to roads where pedestrian traffic is not specifically catered for. Their width does not, therefore, make provision for the mounting of guardrails, or for edge drains or shoulder rounding. The shoulder breakpoint is some distance beyond the edge of the usable shoulder, usually about 0,5 to 1,0 m.

A stopped vehicle can be adequately accommodated by a shoulder which is 3,0 m wide, and there is no merit in adopting a shoulder width greater than this. The shoulder should, on the other hand, not be so narrow that a stopped vehicle would cause congestion by forcing vehicles travelling in both directions into a single lane. However, a partly blocked lane is acceptable under conditions of low speed and low traffic volume. With the narrowest width of through-lane, i.e. 3,1 m, it is possible for two vehicles to pass each other next to a stopped vehicle where the shoulders are not less than 1,0 m wide, giving a total cross-sectional width of 8,2 m to accommodate three vehicles. It is stressed that this width is an irreducible minimum and appropriate only to low lane volumes and low speeds.

Hazards tend to cause a lateral shift of vehicles if located closer than 1,5 m to the lane edge, and for speeds higher than 60 km/h a shoulder width of 1,5 m should be regarded as the minimum.

Intermediate traffic volumes and higher operating speeds require a shoulder width greater than 1,0 m, and three alternative shoulder widths are suggested, namely 1,5 m, 2,0 m and 2,5 m.

The 3,0 m shoulder is appropriate for the highest operating speeds and heavy traffic volumes.

Medians and outer separators

The median is the total area between the inner edges of the inside traffic lanes of a divided road, and includes the inner shoulders and central island. The purpose of the median is to separate opposing streams of traffic and hence reduce the possibility of vehicles crossing into the path of opposing traffic. This is accomplished by the selection of an appropriate median width or by the use of a physical barrier such as a guardrail.

Median width depends not only on traffic volume but also on the function of the road and traffic composition. A median functioning as a pedestrian refuge could be narrower than one protecting a turning vehicle which could be anything up to a combination vehicle (i.e. semitrailer plus trailer). A median narrower than 3,0 m does not offer pedestrians any sense of security, particularly when buses or trucks are travelling in the immediately adjacent lanes.

A median of less than 1,5 m in width is physically dangerous to pedestrians and should not be considered wherever pedestrian traffic is likely to be encountered. However, with severe spatial limitations, it is possible to use medians this narrow. They would serve only to accommodate back-to-back guardrails to ensure vehicular separation. A median that is 5,0 m wide would be able to accommodate a right-turn lane with provision for a pedestrian refuge, but would also require guardrail protection to separate the opposing flows of traffic.

Medians are totally inappropriate in residential streets. These streets are principally directed to the function of accessibility, including vehicles turning right from the street to enter individual properties. Medians, particularly when raised and kerbed, preclude this movement. If medians are depressed, it is possible for vehicles to traverse them but, for oncoming vehicles, this is an unexpected and correspondingly dangerous manoeuvre.

The purpose of an outer separator is to separate streams of traffic flowing in the same direction but at different speeds and also to modify weaving manoeuvres. In general, the standards applied to medians are also appropriate for outer separators. The outer separator could, for example, support the situation of a relatively high-speed road traversing a local shopping area. Vehicles manoeuvring into or out of parking spaces and pedestrians are thus safeguarded from collisions with fast-moving through-traffic.

This situation would, however, constitute poor planning or be forced by a situation outside the control of the planner. This contention refers to the fact that the central high-speed lanes would be a significant barrier to pedestrians wishing to cross the street and effectively create two independent shopping areas from an otherwise integrated unit.

Verges

The verge is defined as the area between the roadway edge and the road reserve boundary.

All facilities not directly connected with the road, e.g. telephone or power lines, are normally located in the verge. In the case of the freeway, the verge is simply the clear space between the shoulder breakpoint and the reserve boundary. On the other hand, in the urban, specifically the residential, environment it is the verge that gives the street its richness and unique character. As in the case of the cross-section as a whole, where the total width is built up as the sum of various disaggregated elements, the verge width is also the sum of the various elements it is required to contain. In general, the verge should have a width of the order of about 5 metres, but, as implied by the preceding statement, this can only be regarded as a very rough rule of thumb.

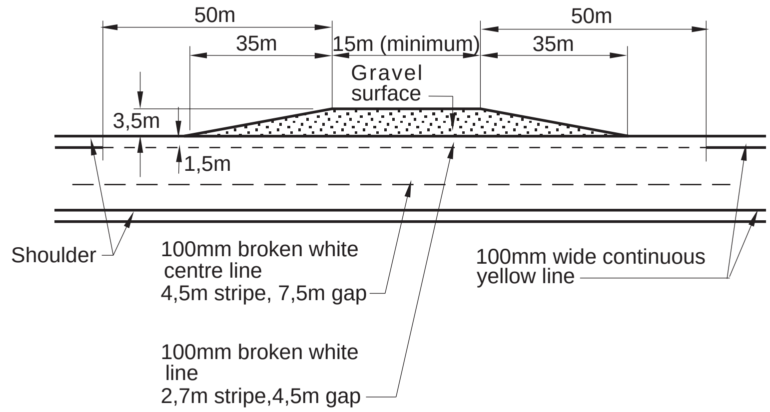
Even where HOV lanes are provided, it is desirable to locate bus stops in the verge. Typical layouts for bus stops are shown in Figure 7.8.

Various elements and their typical widths are listed in Table 7.18.

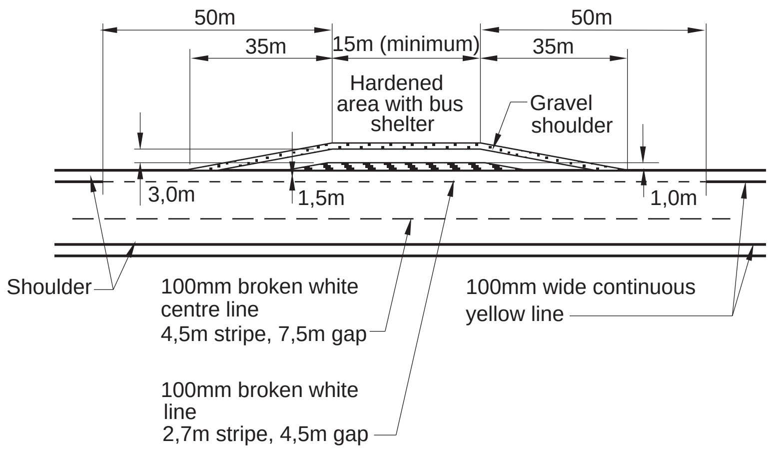
Sidewalks

Wherever there is significant usage by pedestrians, the shoulder is replaced by a sidewalk. A sidewalk is understood as comprising the entire width between the adjacent kerb face and the reserve boundary, and is generally paved over the full width of the verge. If the provision for pedestrians does not use this full width, reference is made to “footways”. The width of the sidewalk is dictated by the anticipated volume of pedestrian traffic, with an additional allowance being made for any other application intended as part of the function of the road reserve.

Reference should be made to the discussion of “hard open spaces” in Chapter 5 of this document.



(a) Gravel surface



(b) Blacktop surface

Figure 7.8: Typical bus stop layouts

Table 7.18: Typical width of verge elements

ELEMENTS	WIDTH (m)
Berm 1,5 m high	6,0
Bicycle paths	1,5 - 3,0
Bus stop embayment	3,0
Bus stop passenger queue	0,7 - 1,4
Clear strip (including kerb and drainage inlet)	2,0
Drainage inlet or manhole	1,5
Driveway approach	5,0
Electric light poles	0,3 - 0,5
Footway (sidewalk)	1,5 - 2,0
Guardrails or barriers	0,5
Kerbs (barrier)	0,15
Kerbs (mountable)	0,3
Kerbs (semi-mountable)	0,15
Landscape strip	3,0
Parking (parallel)	2,5
Traffic signals	0,6 - 1,5
Traffic signs	0,6 - 2,0
Trench width for underground service	1,0 m minimum

The topic of design for pedestrians is exhaustively described in: *Pedestrian facility guidelines: Manual to plan, design and maintain safe pedestrian facilities*. Department of Transport Report 92/126, Pretoria.

In this document, the sidewalk is defined as comprising two elements, these being the "effective" width and the "ancillary" width. The effective width is that portion of the sidewalk dedicated to movement and the ancillary width the portion of the road reserve otherwise used. The effective width is shown as being set back by 0,5 m from the adjacent kerb face.

Required effective widths of sidewalk for various LOS can be calculated by dividing the flow in pedestrians/minute by the values of pedestrians/minute/metre as shown in Table 7.19.

Table 7.19: LOS criteria for sidewalk width

LEVEL OF SERVICE	PEDESTRIANS/MINUTE/METRE
A	0,6
B	2,1
C	3,0
D	4,6
E	7,6

The total width of the sidewalk is thus the effective width as calculated plus the setback of 0,5 m plus the ancillary width determined by the other functions to be accommodated by the sidewalk.

Slopes

Camber and crossfall

Camber implies two slopes away from a central high point, as in a two-lane two-way road, where the cross-section slopes down from the centre line to the shoulders. Crossfall is a single slope from shoulder to shoulder. The slope, whether camber or crossfall, is provided to facilitate drainage of the road surface.

The steepness of slope lies in the range of 2 to 3%. In areas where heavy rainfall is common or where the most economical longitudinal gradient is 0%, the steeper slope is preferred. Cambers steeper than 3% introduce operational problems, both in driving and in increased wear of vehicle components. Where a surfaced shoulder is provided, the camber should be taken to the outer edge of the shoulder. Unsurfaced shoulders should have a crossfall of 4% to ensure a rate of flow across this rougher surface that matches the flow across the surfaced area.

In the case of very narrow reserves, such as in the case of lanes or alleys where spatial restrictions may preclude the provision of drainage outside the width of the travelled way, a negative or reverse camber, i.e. sloping towards a central low point, could be considered. In this case, the centreline of the street is the low point to create a flat V configuration. The entire surfaced width then serves as a drainage area.

Medians

Two different conditions dictate the steepness of the slope across the median: drainage and safety. As suggested earlier, the normal profile of a median would be a negative camber to facilitate drainage. The flattest slope that is recommended is 10%. Slopes flatter than this may lead to ponding and may allow water to flow from the median onto the carriageway.

Slopes steeper than 25% (or 1:4) would make control of an errant vehicle more difficult, leading to a greater possibility of cross-median accidents. If surface drainage requires a median slope steeper than 1:4, this aspect of road safety would justify replacing surface drainage with an underground drainage system.

Cut and fill batters

Gradelines that require cuts or fills so high that their batters require specific attention are alien to mixed-usage streets. The intention with these streets is that the gradeline should be as close as possible to the natural ground level and, preferably, slightly below it. This is necessary to ensure ease of access to adjacent properties and also to support the drainage of the surrounding area.

On vehicle-only roads, the slopes of the sides of the road prism are, like those of medians, dictated by two different conditions. Shallow slopes are required for safety, and a slope of 1:4 is the steepest acceptable for this purpose. The alternative is to accept a steeper slope and provide for safety by some other means, such as guardrails. In this case the steepest slope that can be used is dictated by the natural angle of repose and erodibility of the construction material.

INTERSECTIONS

Introduction

Intersections are required to accommodate the movement of both vehicles and of pedestrians. In both respects, intersections have a lower capacity than the links on either side of them. In consequence, it is the efficiency of the intersections that dictates the efficiency of the network as a whole.

Various measures of effectiveness (MOE) may be proposed. These include energy consumption, time, safety and convenience, and one should not lose sight of the fact that these measures apply as much to pedestrians as they do to vehicles. Furthermore, while they are not mutually exclusive, clashes between these measures can arise. For example, the minimisation of energy usage suggests that the major vehicular traffic should be kept moving at all times. The safety and convenience of all other road users, vehicular and pedestrian alike, would obviously be compromised as a result. Optimisation of the design of any intersection thus requires consideration of the MOEs appropriate to it and to those to whom these measures should be applied.

In the case of residential streets, the needs of pedestrians should take precedence over the needs of vehicles whereas, on higher-order roads, the needs of moving vehicles are more important. It follows that, in the former case, convenience would be a prime measure of efficiency. In the latter, energy consumption becomes significant. In both cases, safety is a major concern.

With regard to vehicular traffic, the operation of an intersection requires that opposing streams of vehicles are forced either to reduce speed or to stop. Optimisation of intersection efficiency in this case refers essentially to a reduction of delay. Delay has two components of interest. In the first, reference is to time costs. Signalisation, for example, forces a major flow periodically to be brought to a stop to allow entry by the minor flow. A signalised intersection will, therefore, always tend to show higher total delay than would a priority-controlled intersection. In the second component, reference is to energy costs because stationary vehicles with their engines idling are still consuming fuel.

Vehicles travelling in a common direction are at a low level of risk. Vehicles travelling in opposing directions are at a higher level of risk. Highest yet is the level of risk associated with vehicles travelling in crossing directions. Most accidents occurring on the road network take place at its intersections.

For reasons both of efficiency and safety it is, therefore, necessary to pay careful attention to the design of intersections. Aspects of design that have to be considered are:

- the location of intersections;
- the form of intersections;
- the type of intersection control; and
- the detailed design of individual intersection components.

These four aspects are discussed in more detail in the sections that follow.

Location of intersections

Two aspects of the location of intersections require consideration. The first of these relates to the spacing between successive intersections and the second to restraints applying to the location of individual or isolated intersections.

Successive intersections

The spacing of successive intersections is essentially a function of planning of the area being served. Minimum distances between intersections are primarily concerned with the interaction between these intersections. In the case of the major links in the movement network, access control measures are usually brought to bear to ensure the efficient functioning of the intersections on them.

Ensuring green wave progression along a route with signalised intersections would require spacings of the order of 500 m.

A driver cannot reasonably be expected to utilise the decision sight distance to an intersection effectively if an intervening intersection requires his or her attention. The sign sequence for an intersection includes signs beyond the intersection. Where an intersection is sufficiently important to warrant a sign sequence, the driver should be beyond the last of these signs before being required to give his attention to the following intersection. Under these circumstances, a minimum spacing of 500 m between successive intersections is also suggested.

Spacings of this magnitude, therefore, typically apply to vehicle-only or to higher-order mixed-usage streets.

On local streets, the goal is maximum accessibility. Any form of access control is inappropriate. One criterion for the spacing of their intersections should be that they are not so close that waiting traffic at one intersection could generate a queue extending beyond the next upstream intersection. Very closely spaced intersections would also result in a disproportionate percentage of space being dedicated to the road network.

Isolated intersections

Considerations of safety suggest various restraints on the location of isolated intersections. The need for drivers to discern and readily perform the manoeuvres necessary to pass through an intersection safely means that decision sight distance, as previously described, should be available on the through-road on both sides of the intersection. The driver on the intersecting road will require intersection sight distance to be able to enter or cross the through-road safely. Modification of the alignment of either the through- or the intersecting road, or of both, may make it possible to meet these requirements for a safe intersection. If not, it will be necessary to relocate the intersection.

The location of an intersection on a horizontal curve can create problems for the drivers on both legs of the intersecting road.

Drivers on the intersecting road leg on the inside of the curve will find it difficult to see approaching traffic, because this traffic will be partly behind them. The fact that a large portion of the sight triangle could fall outside the normal width of the road reserve would also mean that both adequate decision sight distance and adequate shoulder sight distance may be lacking.

Drivers on the leg of the intersecting road on the outside of the curve seldom have any problems with sight distance because, in addition to having approaching traffic partly in front of them, they may have the added height advantage caused by the superelevation of the curve. They do, however, have to negotiate the turn onto the through-road against an adverse superelevation. In the urban situation, where values of superelevation are low, this does not constitute a serious problem.

The risk involved in sharp braking during an emergency should also be borne in mind when locating an intersection on a curve.

Generally, an intersection should not be located on a curve with a superelevation greater than 6%.

The stopping distance required on a downgrade of 6% is approximately 40% longer than that required on a level road. Drivers seemingly have difficulty in judging the additional distance required for stopping on downgrades and it is suggested, as a safety measure, that intersections should not be located on grades steeper than 3%.

If it is not possible to align all the legs of an intersection to a gradient of 3% or less, the through-road could have a steeper gradient because vehicles on the intersecting road have to stop or yield, whereas through vehicles may only have to do so occasionally.

Where steep gradients on the intersecting road are unavoidable, a local reduction in its gradient within the reserve of the through-road should be considered. The reason for this is that buses and freight vehicles have difficulty in stopping and pulling away on steep slopes. Typically, the camber of the through-road would be extended along the intersecting road for a sufficient distance to allow such vehicles to stop clear of the through-lane on the through-road and pull away with relative ease. A distance of approximately 10 m from the shoulder breakpoint is required for this. After that, reverse curves with an intervening gradient of 6% or more can be used to match the local gradeline of the intersecting road to the rest of its alignment. In the case of private accesses, steeper grades can be considered.

One of the consequences of a collision between two vehicles at an intersection is that either or both may leave the road. It is therefore advisable to avoid locating an intersection other than at approximately ground level. Lateral obstructions of sight distance should also be considered when the location of an intersection is being determined.

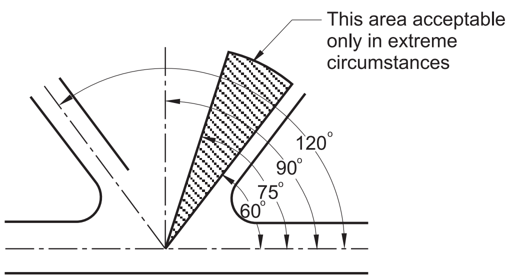
The location of an intersection may have to be modified as a result of an excessive angle of skew between the intersecting roads, i.e. the change of direction to be negotiated by a vehicle turning left off the through-road. Preferably, roads should meet at, or nearly at, right angles. Angles of skew between 60° and 120°, with 0° representing the direction of travel on the through-road, produce only a small reduction in visibility for drivers of passenger vehicles, which often does not warrant realignment of the intersecting road. However, the range of angles of skew between 60° and 75° should be avoided because a truck driver wishing to enter the through-road at an intersection with an angle of skew of 75° or less would find the view to his left obscured by his vehicle. Therefore, if the angle of skew of the intersection falls outside the range of 75° to 120°, the intersecting road should be relocated.

Figure 7.9 illustrates the acceptable angles of skew.

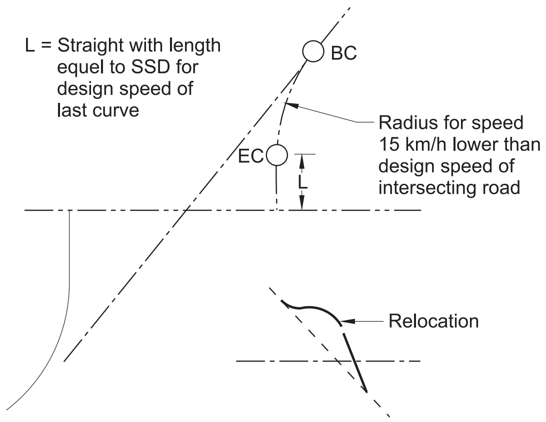
The types of intersection control

Signalisation

Signalisation applies only to higher-order roads and is, in any event, an expensive form of control. Signals should not be employed, in the first instance, as speed-reducing devices. Signals introduce an inefficiency into the system by imposing delay on the through flow to allow the intersecting flow either to cross or to join it. The objective, generally, is to keep the introduced inefficiency to a minimum through the use of proper signal progression. It is interesting to note that an arterial with good signal progression can deliver more vehicles per unit of time to the CBD street system than the latter can handle. This may be an argument in favour of deliberately



(a) Acceptable angles of skew



(b) Relocation of skew intersections

Figure 7.9: Angles of skew

interrupting the flow by a well-planned discontinuity in progression.

Signals should not be used on the highest order roads. At design speeds of 100 km/h and higher, problems with regard to the length of the amber phase and the extent of the dilemma zone manifest themselves. The dilemma zone is that length of road upstream of a signal where, if the signal changes to amber, it is possible neither to clear the intersection before the onset of the opposing green phase nor to stop in advance of the pedestrian cross-walk. If these speeds are to be maintained, freeway operation has to be considered.

Multi-way stop or yield

Reference is to the situation where every approach to the intersection is subject to stop or yield control. Some local authorities in South Africa have also instituted a variation on this form of control by applying stop control not to all but to the majority of approach legs, e.g. two out of three or three out of four. In the United States, the operation of intersections subject to this form of control grants priority to vehicles approaching from the left. The South African operation is based on the principle of “first come, first served”.

This form of control is appropriate to the situation where no clear distinction can be drawn between the intersecting roads in terms of relative importance and where traffic flows on each are more or less equal. It is typically regarded as an interim measure prior to the installation of traffic signals.

Mini roundabouts, traffic circles and gyratories

The principle difference between these three forms of control is the diameter of the central island. The gyratory can have a central island with a diameter of 50 m or more, whereas the traffic circle would typically have a central island with a diameter of the order of 10 m and the mini-roundabout a central island that could range from a painted dot to about 4 m diameter.

The gyratory and the traffic circle operate on the basis of entering vehicles being required to yield to traffic approaching from the right. The mini-roundabout, on the other hand, is controlled by a traffic circle yield sign which, as defined in the Road Traffic Regulations, “Indicates to the driver of a vehicle approaching a traffic circle that he shall yield right of way to any vehicle which will cross any yield line at such intersection before him and which, in the normal course of events, will cross the path of such driver’s vehicle.”

The distinction between behaviour at a traffic circle and that required at a mini-roundabout is not clear to many drivers. Confusion is exacerbated by the fact that a warning sign (a triangular sign with the apex uppermost) requires that right of way should be granted to vehicles approaching from the right as is the case of the traffic circle.

Priority control

Priority control implies that one of the intersecting roads always takes precedence over the other with control taking the form of either stop or yield control. This form of control applies to the situation where it is clear which is the more important of the two intersecting roads. Priority control can also be alternated between successive intersections, for example in a residential area where the layout is more-or-less a grid pattern and the intersecting streets are of equal importance. In this case, the switching of priority would partially serve as a traffic calming measure. In general, this is the most commonly used form of intersection control.

Selection of appropriate control measure

Each intersection should be considered on its own merits and hard and fast rules or a “recipe” method for the selection of the control measure to be employed at any given intersection are not recommended.

Volumes of vehicular traffic being served by the intersection are not considered to be anything other than the roughest of guides. In terms of traffic movement through an intersection, the basic goal should be to minimise delay as far as possible.

Delay has two components: geometric delay and traffic delay. Geometric delay is the delay caused by the existence of the control measure employed. The control measure requires that a driver slow down or stop at the intersection, check that the intersection is clear and then accelerate back to the previous speed. Geometric delay is the difference between the time taken to perform the required set of actions and that expended in travelling through the intersection area at undiminished speed. Traffic delay is that generated by opposing traffic with, as an example, right-turning traffic being impeded by heavy opposing flows or causing impedance to following vehicles that wish to travel straight through the intersection.

Schermers (1987) reports that, at flows of less than 300 vehs/15 mins, traffic delays with four-way stops are less than those with signalisation. In the range above 900 vehs/15 mins, signalisation demonstrated the lowest level of delay. Mini- roundabouts, on the other hand, demonstrate relatively low levels of delay, i.e. even less than four-way stops, up to a flow of about 400 vehs/15 mins. Thereafter, the extent of delay from using mini-roundabouts increases rapidly but remains less than that with signalisation until a flow of 900 vehs/15 mins is achieved. Total traffic delay at 300 veh/15 min amounts to about 3 000 veh.sec/15 mins or about 10 seconds per vehicle on average whereas, at 900 vehs/15 mins, total traffic delay is about 16 000 veh.sec/15 mins or about 18 seconds per vehicle on average.

Priority control does not lend itself readily to analyses of the above form, as the total delay is even more heavily dependent on the split between the opposing flows than in the cases discussed above. Heavy flows on the through-road result in there being relatively few gaps in the traffic stream that are acceptable to drivers wishing to enter or cross from one of the intersecting legs. This results in substantial delays being generated. On the other hand, for the same total flow but with fewer of these vehicles travelling on the through-road, adequate gaps exist for even relatively heavy flows on the intersecting road to experience little delay.

Calculation of delay using a model such as the well-known Tanner formula should be applied to these intersections.

The form of intersections

Intersection form is dictated largely by planning considerations. However, even during the planning phase, due cognisance has to be taken of the safety of vehicles or pedestrians within the area of the intersections. Safety is enhanced by, inter alia, reduction of the number of conflict points at which accidents can occur.

The number of conflict points increases exponentially with the number of legs added to the intersection. A three-legged intersection generates six vehicle-vehicle conflict points, whereas a four- legged intersection has 24 and a five-legged intersection 60. Accident history shows that this increased potential for collision at intersections is, in fact, realised.

In addition to the decrease in safety with an increasing number of approaches to an intersection, there is also a decline in operational efficiency, i.e. an increase in delay.

Multi-leg intersections, i.e. intersections with more than four legs, should not be provided in new designs and, where they occur in existing networks, every effort should be made to convert them to four- or three-legged intersections through channelisation procedures.

Staggered intersections address the problem of skewed intersections (i.e. those with angles of skew outside the limits recommended above) which can be either three- or four-legged. Skewed intersections present a variety of problems. In the first instance, angles of skew greater than those specified generate a line-of-sight problem for the driver on the intersecting road. Secondly, a vehicle required to turn through the acute angle will be moving at a very slow speed, suggesting that those entering the through-road may require a greater sight distance than would be the case for a 90° turn. Finally, the surfaced intersection area becomes excessive. Without channelisation of this movement, a driver traversing the intersection is confronted by a large surfaced area without any positive guidance on the route to be followed. Unpredictable selections of travelled path can represent a distinct hazard to other vehicles in the intersection area.

Four-legged skewed intersections should be relocated so that they form either a single crossing with an angle of skew closer to 90°, or a staggered intersection, which is a combination of two three-legged intersections in close proximity. The right-left stagger, i.e. where the driver on the intersecting road is required to turn right onto the through-road followed by a left turn off it, is preferred. In this case, the driver waits on the intersecting road for a gap in the through-traffic prior to entering the through-road, with the left turn off it being unimpeded except possibly by pedestrians. The left-right stagger may require the vehicle to stop on the through-road while awaiting a gap in the opposing flow to complete the right turn. Relocation of the intersection is to be preferred to the left-right stagger.

Mini-roundabouts can have either three or four approach legs. They comprise either a slightly raised or a painted central island, usually less than 4 metres in diameter, with the traffic lanes being deviated slightly, both to accommodate the island and to force a reduction of speed through the intersection. The discussion above relating to angles of skew applies equally to this form of intersection.

Intersection components

Auxiliary through-lanes

Auxiliary lanes for through-traffic are added outside the through-lanes to match the capacity of the intersection with that of the road between intersections. These lanes are normally only provided at signalised intersections. The length of lane to be added is a matter of calculation. It is dependent on the traffic flow to be serviced and on the length of green time available for the approach leg in question. In the case of priority control, auxiliary through-lanes would not be required on the through-road. Traffic volumes on the intersecting road would probably be too low to warrant their application. This should, however, be checked.

Auxiliary turning lanes

Turning lanes provide for traffic turning either to the left or to the right and can thus be added either outside the through-lanes or immediately adjacent to the centreline.

In the latter case, the through-lanes would have to be deviated away from the centreline if there is not a median island wide enough to accommodate the right-turn lane. Particularly at night, a wet, hence reflective, road surface causes the road markings not to be readily visible. Deviation of a through-lane should therefore be clearly demarcated by road studs to ensure that vehicles do not inadvertently stray into the right-turn lane.

The extent of deviation provided is dependent on the extent of offset provided to the right-turn lane. Good practice suggests that opposing right-turn lanes should be in line with each other to provide the turning driver with the maximum clear view of oncoming traffic that would oppose the turning movement. In this case, the deviation of the through-lane would amount to only half of the width of the turning lane.

Ideally, turning lanes should have the same width as the adjacent lanes but spatial limitations may require that a smaller width be used. The low speeds anticipated in turning lanes in combination with relatively low lane occupancy make it possible to use lesser widths, but the width of the turning lane should not be less than 3,1 m.

The length of turning lanes has three components: the deceleration length, the storage length and the entering taper.

Deceleration should, desirably, take place clear of the through-lane. The total length required is that necessary for a safe and comfortable stop from the design speed of the road. Stopping sight distance is based on a deceleration rate of 3,0 m/s^2 and a comfortable rate is taken as being half this, so that

$$s = \frac{v^2}{38{,}9}$$

where s = deceleration lane length (m)
v = design speed (km/h)

The storage length has to be sufficient for the number of vehicles likely to accumulate during a critical period. It should not be necessary for right-turning vehicles to stop in the through-lane. Furthermore, vehicles stopped in the through-lane while awaiting a change of traffic signal phase should not block the entrance to the turning lane. In the case of unsignalised intersections, the storage length should be sufficient to accommodate the number of vehicles likely to arrive in a two-minute period. At signalised intersections, the required storage length depends on the signal cycle length, the phasing arrangement and the rate of arrivals of right-turning vehicles. The last-mentioned can be modelled using the Poisson distribution which is

$$P(x=r) = \frac{e^{-m}\, m^r}{r!}$$

where e = base of natural logarithms

m = constant equal to the value of arrivals

r = the number of arrivals for which the probability is being calculated.

In both signalised and unsignalised intersections, the length of storage lane should be sufficient to accommodate at least two passenger cars. If buses or trucks represent more than 10% of the turning traffic, provision should be made for storage of at least one passenger car and one bus or truck. It should be noted that the length of the design bus is 12,3 m, compared with the 9,1 m of the truck.

The shorter length could be used only if the road is not intended to serve as a bus route.

Tapers

Tapers can be either passive, allowing a lateral movement in the traffic stream, or active, forcing the lateral movement to take place. Thus, the addition of a lane to the cross-section is preceded by a passive taper, and a lane drop by an active taper. In general, an active taper should be long whereas a passive taper can be short. In the latter case, a taper can be "squared off", meaning that a full-width lane is added instantaneously and the taper demarcated by road marking as opposed to physically constructing a tapered length of road.

Taper rates are shown in Table 7.20.

The lower taper rate for active kerbed tapers is permissible because of the higher visibility of the kerbing which, for this purpose, should be highlighted with paint or reflective markings. Very often, the need for storage space at urban intersections outweighs the need for smooth transitions. In this case the passive taper rate can be reduced to 1:2.

Kerbing and kerb radii

Barrier or semi-mountable kerbing is recommended for intersections because of the more visible demarcation of the lane edge that they offer. In the presence of pedestrians, barrier kerbing is the preferred option. In either case, ramps should be provided for prams and wheelchairs. Kerbing is normally offset by 0,3 m from the lane edge.

Left-turning traffic must be able to negotiate the turn without encroaching on either the adjacent shoulder or sidewalk or on the opposing lane. The latter requirement can, however, be relaxed in the case of the occasional large vehicle on a street with low traffic volumes.

Various forms of edge treatment are described in Table 7.21.

It should be noted that the minimum outer turning radius of a passenger car is of the order of 6,2 m. A passenger car would thus, at a crawl speed, just be able to maintain position relative to a left- turning kerb with a radius of about 4,0 m.

The three-centre curve closely approximates the actual path of a vehicle negotiating the turn. This has the effect of reducing the extent of the surfaced area that has to be provided and is, thus, particularly useful where a change of direction of greater than 90° has to be accommodated. This close approximation to actual wheel paths also suggests that it offers a level of guidance to turning vehicles better than that provided by simple curves.

Corner splays

Depending on the width of the road reserve, it may be necessary to splay the reserve boundary in the intersection area to provide adequate stopping sight distance for drivers both on the major and the minor legs of the intersection. In general, a minimum width of border area around the corner should be of the order of 3,5 m.

Table 7.20: Taper rates

DESIGN SPEED (km/h)	30	40	50	60	80	100
Passive tapers						
Taper rate (1 in)	5	8	10	15	20	25
Active tapers						
Taper rate (1 in) for painted line taper	20	23	25	35	40	45
Taper rate (1 in) for kerbed taper	10	13	15	20	25	30

Table 7.21: Typical edge treatments for left turns

TREATMENT	MINIMUM KERB RADIUS (m)	APPROACH/DEPARTURE TREATMENT
Simple curve	10	Nil
Simple curve with tapers	6	1:15 tapers
Three-centred curve	6	Ratio of curvature 2:1:4
Channelised turning roadway with three-centred curve	15	Ratio of curvature 2:1:4
Channelised turning roadway with simple curve and tapers	25	1:10 tapers

Channelisation

Channelisation involves the use of islands and road markings and is usually required where traffic volumes are high. The purposes of channelisation with regard to vehicle movement are to:

- separate areas for manoeuvring and present drivers with one decision at a time;
- control the direction of movement of vehicles to obtain small angles for merging and diverging at low relative speeds or approximate right angles for crossing at high relative speeds;
- control speed by redirection or funnelling, the latter implying a steady reduction of lane width over a short distance;
- provide protection and storage for turning vehicles;
- eliminate excessive surfaced areas which permit drivers to perform improper manoeuvres or to travel along paths unpredictable to other drivers;
- prevent illegal manoeuvres, such as turns in the wrong direction into one-way streets; and
- provide space and protection for traffic control devices and other road signs.

Channelisation is also required at intersections where traffic volumes may be relatively low but pedestrian volumes high. In this case, the function of channelisation is directed towards providing refuge for pedestrians seeking to cross the various traffic flows.

Walking speeds are typically of the order of 1,5 m/s. In the vicinity of old-age homes and similar areas, accommodation should be made for a walking speed of about 1 m/s. Crossing a two-lane street would thus require a gap or a lag of about seven seconds. A gap is the difference between the times of arrival at the crossing point by two successive vehicles. The lag is the unexpired portion of a gap, i.e. the time between the pedestrian arriving at the crossing point and an opposing vehicle arriving at the same point. On multilane streets, the crossing time is correspondingly higher.

If traffic flows are such that a gap or lag equal to or greater than the crossing time is not available at about one minute intervals, pedestrians are tempted to cross by pausing on the roadmarkings between the various flows to await the gap in the next flow to be crossed.

This is not a normally recommended practice. It can be avoided by the provision of islands which have the effect of reducing the duration of the required gap, hence increasing the probability of its occurrence. Alternatively, the creation of adequate gaps can be forced by the use of demarcated or, preferably, signalised pedestrian crossings.

Islands

Islands, whether painted or kerbed, can be classed into three groups:

- directional islands, which direct traffic along the correct channels or prevent illegal manoeuvres;
- divisional islands, which separate opposing traffic flows; and
- refuge islands, to protect pedestrians crossing the roadway or turning vehicles that are required to stop while awaiting gaps in the opposing traffic, and also to provide space for traffic control devices.

Typically, islands are either long and narrow or triangular in shape. The circular central island is considered more a traffic control device than a channelisation feature. Small islands have low visibility and cannot serve safely as either accommodation for pedestrians or traffic control devices. Islands should not have an area of less than 5 m^2 or width of less than 1,2 m. Islands used as pedestrian refuges should preferably be 3,0 m wide. Painted islands do not constitute a significant refuge for pedestrians and should not be used in this application. Pedestrian refuge islands should be provided with barrier kerbing and ramps for wheelchairs and prams.

Island kerbing is usually introduced suddenly. For this reason, the approach end requires careful design. In the case of triangular islands, the point of intersection of the approach sides of the island should be rounded and painted and, possibly, also be provided with reflective markings. The approach end should also be offset from the edge of the adjacent lane, as shown in Figure 7.10.

Turning roadways

The normal track width of a vehicle is the distance between the outer faces of the rear tyres. When a curve is being negotiated, the rear wheels track inside the front wheels and the track width then becomes the radial distance between the path of the outside front wheel and the inside rear wheel. On turning roadways, it is this greater width that has to be accommodated. The turning roadway width is thus a function of:

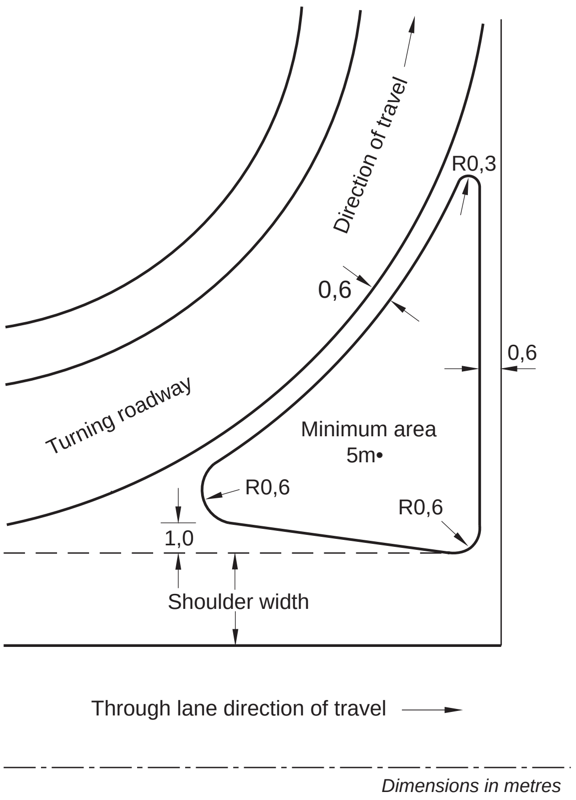


Figure 7.10: Layout of island

- the flow operation being designed for;
- the design vehicle to be accommodated; and
- the radius of curvature of the turning roadway.

With regard to the first-mentioned, three operations need to be addressed. These are:

- Case I: One-lane one-way operation, with no provision for passing a stalled vehicle.

- Case II: One-lane one-way operation, with provision for passing a stalled vehicle.

- Case III: Two-lane operation, either one-way or two-way.

Design vehicles to be accommodated are passenger cars (P), single-unit trucks or buses (SU) and articulated vehicles (WB12). The selection of design vehicle is addressed as the traffic condition being designed for. These are:

- Traffic Condition A - Predominantly P vehicles but some consideration given to SUs.

- Traffic Condition B - Sufficient SUs (approx 10%) to govern design but some consideration given to WB12s.

- Traffic Condition C - Sufficient WB12s to govern design.

Provision for passing stalled vehicles and two-lane operation require a combination of design vehicles. The combinations normally considered are shown in Table 7.22.

Table 7.22: Design traffic conditions

CASE	A	B	C
I	P	SU	WB12
II	P-P	P-SU	SU-SU
III	P-SU	SU-SU	WB12-WB12

The selected lane width either should or can be modified depending on the selected edge treatment. Three edge treatments have to be accommodated. These are mountable kerbing, barrier or semi-mountable kerbing, and the stabilised shoulder. They can occur in combination, e.g. a barrier kerb on one side of the lane and a stabilised shoulder on the other. Shoulders would only be used at intersections if the cross-section of the road upstream and downstream of the intersection includes shoulders. In the presence of pedestrians, kerbing is necessary.

Recommended turning roadway widths are listed in Table 7.23. for the various cases, conditions and edge treatments.

The crossfall prevailing on the through-lanes is simply extended across the turning roadways. Should superelevation be deemed to be necessary, it could be achieved by the use of a crossover crown line, in which case the maximum superelevation could be as high as 6%.

Mini-roundabouts

Mini-roundabouts are the one exception to the general practice of avoiding the use of intersection controls as traffic calming devices. They can be used either as intersection controls or as traffic calming devices. In the latter case, however, their application should be as part of area-wide traffic calming schemes rather than in isolation.

The mini-roundabout comprises a central circular island, deflector islands on each of the approach legs and a circular travelled way around the central island. On a three-legged or T-intersection, it is frequently necessary to apply speed humps to the approach from the left on the cross leg of the T. The reason for this is that the location of the circular island is invariably such that traffic on this leg has a straight line path through the intersection and it is necessary to force a reduction of speed.

Table 7.23: Turning roadway widths (m)

RADIUS ON INNER EDGE (m)	CASE I			CASE II			CASE III		
	DESIGN TRAFFIC CONDITION								
	A	B	C	A	B	C	A	B	C
15	5,4	5,4	6,9	6,9	7,5	8,7	9,3	10,5	12,6
25	4,8	5,1	5,7	6,3	6,9	8,1	8,7	9,9	11,1
30	4,5	4,8	5,4	6,0	6,6	7,5	8,4	9,3	10,5
50	4,2	4,8	5,1	5,7	6,3	7,2	8,1	9,0	9,9
75	3,9	4,8	4,8	5,7	6,3	6,9	8,1	8,7	9,3
100	3,9	4,5	4,8	5,4	6,0	6,6	7,8	8,4	0,0
125	3,7	4,5	4,8	5,4	6,0	6,6	7,8	8,4	8,7
150	3,7	4,5	4,5	5,4	6,0	6,6	7,8	8,4	8,7
TANGENT	3,7	4,5	4,5	5,1	5,7	6,3	7,5	8,1	8,1

WIDTH MODIFICATION APPROPRIATE TO EDGE TREATMENT			
Mountable kerb	none	none	none
Barrier kerb one side	add 0,3 m	none	add 0,3 m
Barrier kerb both sides	add 0,6 m	add 0,3 m	add 0,6 m
Stabilised shoulder one or both sides	Condition B & C lane widths on tangent may be reduced to 3,7 m for 1,2 m or wider shoulder	Deduct shoulder width; minimum width as for Case I	Deduct 0,6 m where shoulder is 1,2 m or wider

Mini-roundabouts have negative implications for cyclists and, to a lesser extent, for pedestrians. Circulation of traffic through a roundabout is not straightforward and the need for vehicles to stop at the intersection is reduced. Crossing opportunities for pedestrians are thus similarly reduced and the task of judging acceptable gaps is more difficult. The circulatory flow and reduced carriageway width offer less protected space for cyclists. In addition, motorists are seldom prepared to yield the right of way to cyclists. These two classes of road users thus require careful consideration in the design of these intersections and appropriate facilities provided for them.

Central island

The central island is typically of the order of 4,0 m in diameter. It may simply be a painted island, although the preferred option is that it should be an asphalt hump. The latter offers more specific guidance to drivers and ensures that the mini-roundabout operates as intended. The height of the hump should be in the range of 75 mm to 100 mm. This is a compromise between the height that is visible to approaching drivers and the height that long vehicles can traverse without damage. The guidance role is strengthened, and the asphalt hump simultaneously protected from damage by passing vehicles, if the central island is surrounded by a 25 mm high steel hoop, securely anchored to the road surface.

Width of travelled way

The inscribed circle diameter is dependent on the design vehicle. Britain and Australia both recommend a diameter of 28 m, suggesting thus that the travelled way has a width of 12 m, assuming that the traffic circle has a diameter of 4 m. This corresponds to use of a WB-12 as the design vehicle. South African practice, where the mini-roundabout is typically used in residential areas, is to use the passenger car or the bus as the design vehicle of choice. If the roundabout is not located on a bus route, the inscribed circle diameter could be reduced to 14 m. Bus routes would require an inscribed circle 25 m in diameter and this width is adequate for two-lane operation by passenger cars.

Deflector islands

Kerbed islands are used to guide vehicles into the intersection area. The side of the island closest to the traffic approaching the intersection thus constitutes an active taper. A taper rate of 1:10 appropriate to a design speed of 30 km/h should be employed. The side of the island downstream of the intersection is also an active taper insofar as it serves as a transition from the wider lane width around the mini-roundabout to the normal lane width applying to the rest of the street, and the same taper rate applies. In this case, however, the taper rate is relative to the direction of movement of vehicles exiting from the intersection. The approach end of the island should be rounded and offset as illustrated in Fig 7.10.

The kerbed island is normally preceded by a painted island.

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Chapter 8

Roads: Materials and construction

8

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

This chapter focuses on

- the philosophy of residential street design, with particular emphasis on the need for a shift from a previous approach of risk elimination to an approach of risk management;
- the need for layout planning and drainage design to be considered before the structural design of the street is addressed;
- the relative importance of the street and the level of service expected by the users and provided by various road types;
- the structural design of street pavements;
- approaches to construction that include conventional methods, labour-intensive methods and approaches that allow small contractors to participate;
- practical guidelines for the maintenance of streets;
- the comparison of proposed pavement designs on a life-cycle cost basis; and
- references to other guidelines for additional information.

SCOPE AND NATURE OF THIS CHAPTER

The chapter covers issues ranging from general design philosophy to detailed methods for the structural design of street pavements. The overall design philosophy, design approach and design procedures presented in this chapter are applicable to urban streets in South Africa, and may be used for similar applications in the sub-region.

DESIGN PHILOSOPHY

By nature, structural design of street pavements tends to be prescriptive owing to the restrictions imposed by geology, topography, design traffic and materials. The use of catalogues in the past (even though intended as design examples) has perhaps also given rise to the view that these options are set. In order to make this section of the guidelines more facilitative than prescriptive, engineering options - together with the implications of choosing an option - are presented.

The structural design of the pavements in an urban environment cannot be done in isolation. Funding mechanisms, the ability to execute maintenance when required, the ability of the end users to pay, appropriate standards and environmental impact will influence the structural design process. These issues cannot be quantified numerically in the design process but should rather determine the mindset of the designer. The designer must therefore exercise his own judgement during the design process, taking into consideration the above issues and the risk associated with his design decisions.

Provision and ability to pay

Street upgrading and maintenance in residential areas is traditionally funded from rates levied and collected by the local authority. The level of service provided will need to take into account this source of funding and an appropriate level should be found which will be affordable to the ratepayers.

Appropriate standards

In the past, similar guideline documents have focused on the more highly developed component of South Africa with an emphasis on standards appropriate to high levels of car ownership and high traffic volumes. Standards have also been very conservative, with the use of low-risk pavements with concomitant high construction cost. A shift in emphasis has occurred and service provision to the whole spectrum of development levels now needs to be considered. Due to the differing traffic volumes that may be expected at different levels of development and the need to move towards risk management, standards appropriate to particular applications must be considered. It should be noted that “appropriate standards” does not necessarily imply “relaxed standards”.

Environmental impact

Roads (and transport routes in general), by their nature, can be environmentally intrusive. It is important, therefore, to construct them so that their impact on the environment is as small as possible.

Comparatively little work, other than certain specific environmental impact assessments, has been conducted on the impact of roads and the associated traffic on the environment. It is therefore considered important that, before any new roads are constructed, or existing roads are rehabilitated or upgraded, the relevant authorities determine the impact on both the biophysical and the socio-economic environments. In Africa, most funding agencies will not consider the granting of loans for road development and upgrading until a thorough investigation has been concluded.

A number of impacts associated with the construction, maintenance and use of roads have been identified and are summarised below. The list is not exhaustive but appraises most of the major socio-economic and biophysical considerations, and includes impacts on the:

- physical characteristics of the site and surroundings;
- ecological characteristics of the site and surroundings;
- current and potential land use and landscape character;
- cultural resources;
- socio-economic characteristics of the affected public;
- adjacent and associated infrastructure services;
- social and community services and facilities;
- the nature and level of present and future environmental pollution; and
- health and safety.

Risk

In order to provide street infrastructure in residential environments where there invariably are budget limitations, urban authorities need to adopt a philosophy of risk management. Where a local authority is responsible for the costs of construction and maintenance, the philosophy of structural design must move towards an approach that balances construction and maintenance costs by sensible risk-taking and risk management. This document, therefore, allows for the current need for risk management and has moved away from previous approaches of "designing out" risk by using high-cost street pavements.

There will, of course, be areas where high construction costs are borne by the property developer or purchaser who demands a higher standard of street than might be economically justifiable. In such a case the benefit of lower future maintenance costs is passed on to the local authority and free-market conditions will determine the ultimate standard of construction.

There are currently certain streets built at low cost with associated high risk, and this document recognises this fact. In informal settlements there may be unknown and unclassified street systems, and arterial routes may be gravelled. A risk has therefore been assumed on these informal streets, which may have been taken over by a local authority. It is also possible that an authority could build a street structure at some risk and let traffic and time show the problem areas.

In order to address the issue of risk associated with the use of a particular pavement type in a particular street category, the link between structural design, maintenance and construction is addressed in the section on life-cycle costs. The document allows for risk management by stating the risk involved in using alternative designs. This is achieved by "qualitative statements" regarding the risk and a list of risk-management approaches that can be used to manage higher risk structures.

The greater the relaxation in specification, the greater the risk. In order to manage this and control consequences, only one parameter should be relaxed at any one time. For example, if the drainage is inherently poor and cannot be cost-effectively rectified, no attempt at reducing material quality or pavement thickness should be considered.

Risk must be related to road usage. Higher risk can be accommodated on lower-usage roads, while the inverse is true for main routes and arterials.

THE PAVEMENT DESIGN PROCESS

The aim of structural design is to produce a structurally balanced pavement which, at minimum present worth of cost, will carry the traffic for the structural design period in the prevailing environment, at an acceptable service level without major structural distress. If necessary, the pavement should be capable of being strengthened by various rehabilitation measures to carry the traffic over the full analysis period.

This aim is achieved by protecting the subgrade through the provision of pavement layers. However, issues such as level of service, stormwater accommodation, traffic, pavement materials, subgrade soils, environmental conditions, construction details and economics are all part of the process.

It is important to attend to layout planning and drainage design before the structural design of the street is addressed, as these will lend better definition to the role of the street in the larger development, and also affect the final structural design of the pavement.

Level of service

The "level of service" of streets is expressed as a combined function of the pavement and stormwater drainage elements. A chosen level of service has to be achieved at the lowest possible life-cycle cost.

Stormwater accommodation

The importance of topography on the structural design and functional use of streets is clearly reflected in the drainage and maintenance requirements of streets in general. Macro-drainage is relevant to this discussion. Streets that cross contours at an angle, or even perpendicularly, pose the most drainage problems. In such cases function rather than structure may require that a street be paved or provided with erosion protection. It is therefore important that requirements described in the chapters on layout planning (Chapter 7) and stormwater management (Chapter 6) be met before one embarks on the structural design.

Unpaved streets

In rolling and mountainous terrain there may be steep gradients which result in the erosion of gravel streets and, in particular, erosion of their drainage facilities, with direct implications for their safety and functional use. A longitudinal street gradient of 5% is an average value above which erosion problems may occur on unpaved streets, and slopes steeper than this would warrant additional attention. Gravels in the upper range of the suggested plasticity index (PI) could effectively reduce erosion, but local conditions should be considered in the detailed evaluation.

Piped systems vs surface systems

The use of the road surface - or of surface channels - to accommodate the minor stormwater flows can be more appropriate than the use of piped systems in certain instances, provided safety is not compromised. Areas under development and areas where verges are not grassed can give rise to high silt loads in the stormwater flows, which can rapidly block piped systems. The dumping of refuse and other debris into stormwater inlets and manholes is a common occurrence in some residential areas and this will also lead to blockages. In areas where regular maintenance of piped systems does not take place, surface systems are probably more appropriate.

Economic considerations and design strategy

Life-cycle costs

Providing a traffic circulation network to a residential area involves both construction (capital) and maintenance (operating) costs. For a local authority which is responsible for these functions, the costs of both construction and maintenance must be minimised to provide a service at the lowest total outlay. The total life-cycle cost of a road should include the vehicle operating costs, but these can be disproportionally high and are often neglected. In this case only the agency cost is evaluated.

Upgrading and staged construction

Two concepts that need to be considered as part of the life-cycle strategy of a street during design, are “staged construction” and “upgrading”. Although it is difficult to exactly define and completely separate these concepts, some characteristics may be more typical of one than of the other.

The aim of staged construction is to spread the financial load from the initial construction period to some stage later during the life cycle of the facility. On the other hand, upgrading will normally take place when the demands on an existing facility far exceed the level of service the facility can provide. The influence of doubling the contractor’s establishment costs needs to be evaluated carefully when staged construction is considered.

STRUCTURE OF THIS CHAPTER

The flow diagram in Figure 8.1 outlines the pavement design process. The components may not all be clear at this stage, but will be discussed in detail in the following sections.

This discussion will be done in two parts:

- the components of the process will be discussed in general; and
- a detailed discussion on the design process for each of the five branches in the diagram will follow.

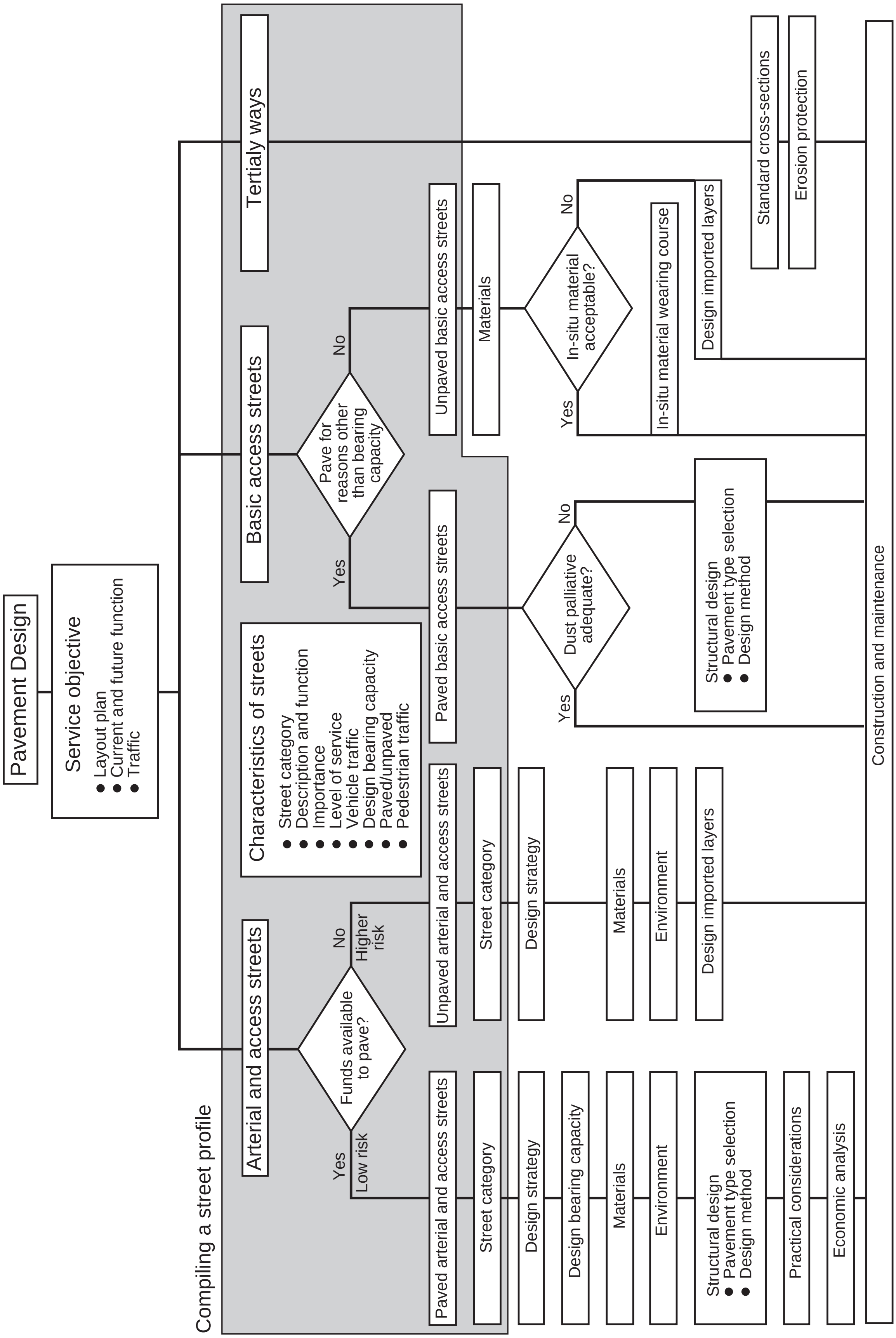


Figure 8.1: Street pavement design flow diagram

THE COMPONENTS OF THE DESIGN PROCESS

SERVICE OBJECTIVE

Before the designer starts in earnest with the actual pavement design of a particular street, he must consider the background against which to design and the objective of providing the facility. This does not need to be considered in great detail at this early stage of the design, but it will guide the direction of the design process. Factors that need to be considered include the layout and drainage plan, the current and future functions of the street and the anticipated traffic. The layout and stormwater drainage plan should be well defined at this stage, which also defines the function of the street to some extent. Detailed calculations on design traffic will be done at a later stage in the design, and only an estimate of the number of vehicles is required at this stage.

COMPILING A STREET "PROFILE"

The shaded area in Figure 8.1 highlights the major decision-making part of the design process. A decision has to be made at this stage on selecting an appropriate street "profile", which will determine the design procedure to adopt. A number of characteristics define the profile of the street. These include the street category, the description and function of the street, the importance of the street, the level of service of the facility (street and drainage combined), vehicle traffic, design bearing capacity, street standard (paved/unpaved) and even the pedestrian traffic expected.

Street categories

Typical street characteristics are listed in Table 8.1. For the purpose of this document, four different street categories, namely UA, UB, UC and UD, are considered. These categories range from very important arterial

Table 8.1: Typical street characteristics

STREET CATEGORY		UA	UB	UC	UD
		ARTERIAL STREETS		ACCESS STREETS	
Description and function		Vehicles only	Higher-order mixed pedestrian and vehicle route	Lower-order mixed pedestrian and vehicle route	Pedestrian and vehicle access route
Level of service (LOS)		High LOS	Moderate LOS	Moderate to low LOS	Low LOS
Traffic (vehicles per day)		>600	<600 (paved) <350 (unpaved)	>75	<75 <5 heavy vehicles
Traffic (no. of E80s)*	(a) If street carries construction traffic	1-50 x 10^6 E80s per lane	0,3-3 x 10^6 E80s per lane	<0.3 x 10^6 E80s per lane	
	(b) If street does not carry construction traffic	1-50 x 10^6 E80s per lane	0,03-3 x 10^6 E80s per lane	<0,3 x 10^6 E80s per lane	
Standard		Most likely paved	Paved/unpaved	Most likely unpaved, or paved for reasons other than traffic	Paved for reasons other than traffic
Pedestrian traffic		None	Very little, controlled	High, controlled	High, uncontrolled

* E80s: Equivalent 80kN single axle loads

streets with a very high volume of traffic, to less important lightly trafficked residential streets.

Street function

Arterial streets

Arterial streets are the major routes providing mobility between and within residential, recreational, commercial and industrial areas. The traffic volumes on these streets will be high and the streets will generally carry significant numbers of buses and other heavy vehicles.

Access streets

Access streets include all residential streets below the level of urban bus routes. Their primary function is access to residential erven and they will thus carry few heavy vehicles. Depending on the level of development and affluence of the area, traffic may be so light that the primary structural design objectives may relate more to minimising damage from erosion than to supporting the traffic.

Access streets may be further divided into access streets with traffic levels of more than 75 vehicles per day and those with less than 75 vehicles per day, of which less than 5 are heavy vehicles. Basic access streets would generally be found in developing areas where vehicle ownership is low.

Level of service (LOS)

The level of service that a user expects from a street is related to the function of the street, to the general standard of the facility and partly to the volume of traffic carried. For example, the user will expect a better riding quality on a dual-carriageway arterial street than on a minor residential street. Irrespective of this, the user will generally expect the highest possible standard.

Streets in a residential area function at different levels of service (LOS). The LOS values are determined by the functional use of the street, the level of construction and the drainage provision. In Table 8.2 the description of each type of street and its drainage is given, with the associated LOS values. In the matrix the combined LOS value is given. The final LOS value is determined mostly by the LOS of the drainage.

Street standards

Arterial streets

Paved arterial streets:
The heavy traffic carried by arterial streets will generally justify their being built to a traditional paved standard, if funds are available.

Unpaved arterial streets:
Although unpaved arterial streets exist in many areas, it is almost impossible to justify them on an economic or social basis. The cost of maintaining them is excessive and the frequency of maintenance necessary to retain a riding quality of any acceptability results in safety hazards. In addition, the user costs, in comparison with a paved street, are extremely high. All attempts should be made to upgrade unpaved arterials carrying bus and heavy goods vehicles to a suitable, relatively low-risk, paved standard as rapidly as possible, whilst arterials carrying mostly light goods vehicles can afford slightly higher-risk pavements.

Although undesirable, financial constraints may prevent an authority being able to pave all arterial streets in a network. The street network should then be prioritised on an economic basis to identify candidate streets for upgrading. The optimum use of available materials is thus necessary to reduce the undesirable properties as far as possible.

Access streets

A distinction is made between access streets (>75 vehicles per day) and basic access streets (<75 vehicles per day). Access streets carrying more than 75 vehicles per day require a more robust design than those carrying lighter traffic volumes (basic access streets).

Paved access streets:
It may be economically justified to pave access streets at levels of 75 vehicles per day and above. The aspirations of residents and street users may also influence the decision.

Unpaved access streets:
Access streets with relatively light traffic can justifiably be left unpaved. If the street is to have an unsealed pavement it is essential that the best quality material available locally is used (blending and/or processing should be considered where necessary to improve the materials), and some method of dust palliation could be considered.

Basic access streets

Basic access streets are defined as access streets in the early stages of development, and are designed and constructed subject to constraints imposed by the administrative authority owing to limited budgets or other circumstances. The objective is to maximise the performance of such streets within these constraints.

Designs for such urban residential streets with low traffic volumes (up to 75 vehicles per day of which

Table 8.2: Levels of service (LOS) of streets or drainage and combined facilities

STREET CATEGORY	LOS	DRAINAGE LOS	4 & 5	3	2	1
	Street LOS	Description	Pipe system, kerbs, gutter and surfaced street	Lined channel on shoulder or on street	Unlined channel on shoulder or on street	Unsurfaced street with provision of drainage with sheet flow
UA	**5**	Primary streets (bus routes) with a designed structure and surfacing, or	**5**	**5**	**4**	
	3	Gravel		**3**	**2**	
UB	**5**	District and/or local distributors (bus routes) with a designed structure and surfacing, or	**5**	**5**	**3**	
	3	Gravel		**3**	**2**	
UC	**5**	Residential access collectors with a designed light structure and surfacing, or	**5**	**4**	**3**	**2**
	3	Gravel, or		**3**	**2**	**1**
	1	In situ		**3**	**2**	**1**
UD	**5**	Access and basic access streets with surfacing on light structure, or	**5**	**4**	**3**	**2**
	3	Gravel, or		**3**	**2**	**1**
	1	In situ		**3**	**2**	**1**

up to five are heavy vehicles) make maximum use of in situ material and require realistic material standards.

Paved basic access streets:
Factors determining the acceptable performance of such light-structure streets are

- drainage (surface and subsurface);
- material quality;
- construction control;
- control of overloading; and
- maintenance.

The philosophy of basic access streets is to concentrate on the engineering impact of drainage, material quality and construction control, to achieve more realistic minimum standards for the circumstances described earlier. Although cost comparisons are not dealt with separately, the principle is that cost-saving is maximised.

Unpaved basic access streets:
These are the lowest quality basic access streets acceptable. Although undesirable from the point of view of dust generation, the production of mud when wet, and the need for constant maintenance, financial constraints may preclude the use of higher quality streets for the very low traffic typical of these streets. The optimum use of available materials is thus necessary to reduce the undesirable performance properties as far as possible.

Earth basic access streets are considered a viable option only if the in situ material is of a quality that will support vehicles even in a soaked condition. One of the major problems with earth streets is that they initially form tracks, and, if bladed, a street that is lower than the surrounding terrain results, with significant drainage problems.

Tertiary ways

Unpaved basic access streets may evolve from informal access routes, which are referred to as tertiary ways in this document.

In developing areas an infrastructure of narrow ways which carry no or few vehicles exists. These non-trafficked tertiary ways are mostly informal and unserviced, but in older developing areas they are formalised (upgraded) by the provision of surfacings, drainage or even services like water and electricity.

In an informal residential development no layout planning for tertiary ways is done. The existence of these ways is dictated by pedestrians' needs to follow the shortest possible route. It is advisable to design for tertiary ways during layout planning as these form the basic links between dwellings. In a formal design this facility can enhance the design principles applicable to the higher order of streets.

Summary of street standards

The street standards that would generally be considered for application to particular street categories are summarised in Table 8.3.

The street as public open space

In the lower street categories (access collector and, particularly, basic access streets) the street functions are less the preserve of the motor vehicle and more the preserve of the pedestrian and resident. This is particularly true of developing areas, where the incidence of traffic is likely to be low. This may be recognised by the provision of a smooth surface.

Streets may also function as firebreaks in areas of high housing density - a possible justification for retaining wide street reserves in some instances.

DESIGN STRATEGY

Paved streets

The design strategy could influence the total cost of a pavement structure. Normally, a design strategy is applicable only to paved Category UA and UB streets. For unpaved streets, or paved Category UC and UD streets, the design periods are fixed.

The analysis period (see Figure 8.2) is a convenient planning period during which complete reconstruction of the pavement is undesirable. The structural design period, on the other hand, is defined as the period for which it is predicted with a high degree of confidence that no structural maintenance will be required.

In order to fulfil the design objective of selecting the optimum pavement in terms of present worth of cost,

Table 8.3: Categorisation of street standards

ARTERIAL STREETS (UA AND UB)		ACCESS STREETS (UC AND UD)					
Paved collector/ dis-tributor streets (bus routes)	Unpaved collector/ distributor streets (bus routes)	Access streets (>75 vehicles per day)		Basic access streets (<75 day; <5 heavy)			
		Paved access streets	Unpaved access streets	Paved basic access streets	Unpaved basic access streets		
					Gravel basic access streets	Earth basic access streets	Tertiary ways

it is necessary to consider how the pavement is expected to perform over the entire analysis period. The manner in which a design strategy can be presented is demonstrated schematically in Figure 8.2, which shows the generalised trends of riding quality decreasing with time, and traffic for two different pavement structures, namely:

Design 1, which requires resealing to maintain the surface in a good condition, and later some structural rehabilitation such as an overlay (Figure 8.2 (a)); and
Design 2, which is structurally adequate for the whole of the analysis period and requires only three resealings (Figure 8.2 (b)).

It is important to note that any design procedure can only estimate the timing and nature of the maintenance measures needed. Naturally, such estimates are only approximations, but they provide a valuable guide for a design strategy. The actual maintenance should be determined by a proper maintenance procedure. The accuracy of the prediction could be improved by having a feedback system.

Selection of analysis period

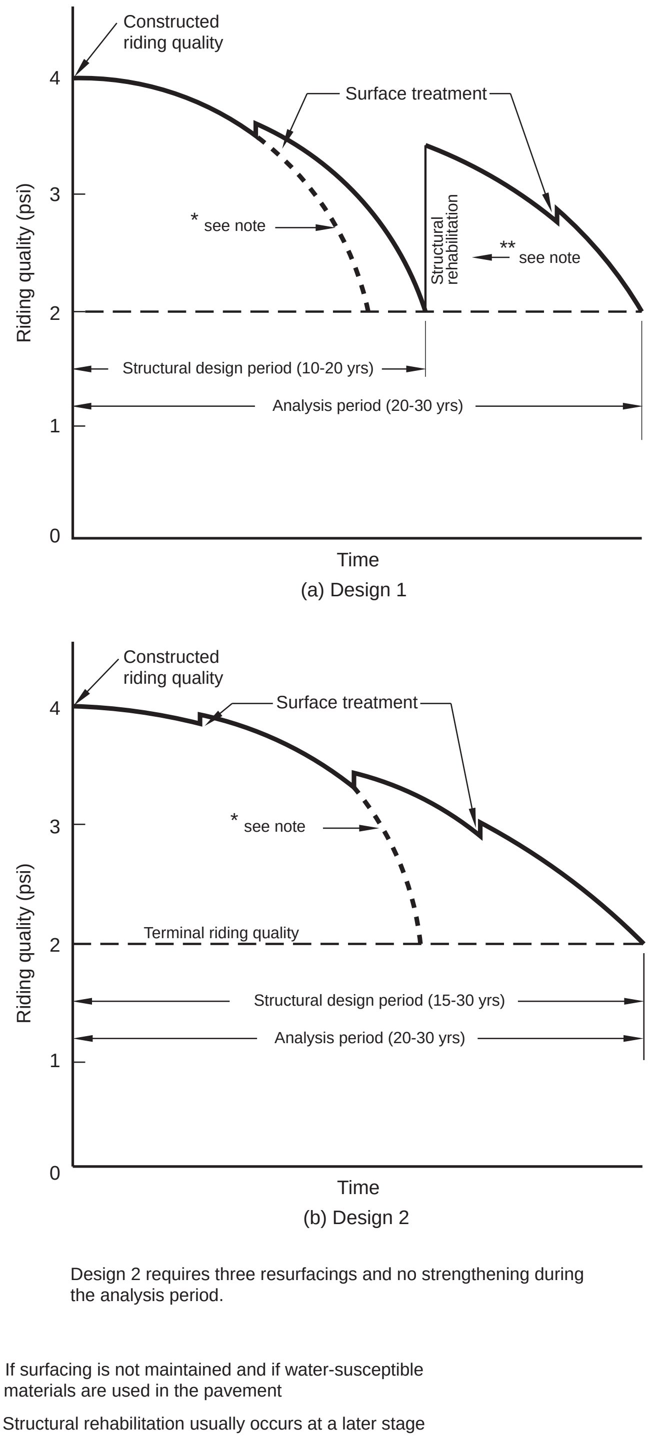


Figure 8.2: Illustration of design periods and alternative design strategies

The analysis period is a realistic cost period. There may be a difference between the analysis period and the total period over which a facility will be used. The analysis period is often related to the geometric life. If the street alignment is fixed, a period of 30 years should be used. In the case of a short geometric life in a changing traffic situation, a shorter analysis period should be used.

Structural design period

Selection of structural design period

Table 8.4 gives a summary of suggested structural design periods for different road categories.
Category UA streets

Table 8.4: Structural design periods for various street categories

STREET CATEGORY	STRUCTURAL DESIGN PERIOD* (YEARS)	
	RANGE	RECOMMENDED
UA	15-25	20
UB	10-25	20
UC	10-30	20

* The analysis period for category UA and UB streets is 30 years.

Category UA streets

For Category UA streets, the structural design period should be reasonably long because

- it is usually not politically acceptable for street authorities to carry out heavy rehabilitation on recently constructed pavements;
- street user costs are high and the cost of the disruption of traffic will probably cancel out any pavement cost savings that result from the choice of short structural design periods; and
- the street alignment is normally fixed.

Category UB streets

For Category UB streets, the structural design period may vary, depending on the circumstances. Long structural design periods (20 years) will be selected when circumstances are the same as for Category UA streets.

Factors that encourage the selection of short structural design periods are

- a short geometric life for a facility in a changing traffic situation;
- a lack of short-term funds; and
- a lack of confidence in design assumptions, especially the design traffic.

Structural design periods may range from 10 to 25 years. Normally a period of 20 years will be used (Table 8.4).

Category UC streets

For category UC streets (residential streets) a fixed structural design period of 20 years is recommended (Table 8.4).

Category UD streets

The traffic volume is so limited that no structural design period is applicable.

DESIGN TRAFFIC AND BEARING CAPACITY

Paved streets

Real life traffic consists of vehicles spanning a range of axle loadings. The composition of the traffic on one street will differ from that on another because of the differences in street category and function. The unique traffic spectrum on a particular street will be the design traffic for that street.

Pavements are, however, designed for a number of *standard 80 kN axles* (SA standard axles). The total number of standard axles that a pavement structure will be able to carry over its design life is referred to as the bearing capacity of the pavement. The design bearing capacity of a pavement is therefore considered to be constant for a particular pavement structure (ignoring environmental effects). It is customary to design pavements for a bearing capacity interval rather than a specific value. These intervals are referred to as pavement classes and Table 8.5 summarises pavement design classes based on pavement bearing capacity. This is done because of the large variation associated with real pavement performance and the associated difficulty in predicting pavement life.

Although the bearing capacity of a pavement is considered to be constant, different traffic spectrums will have different damaging effects on the pavement due to the different axle-load compositions of the traffic spectrums. The cumulative damaging effect of all individual axle loads is expressed as the number of *equivalent 80 kN single-axle loads* (ESA, equivalent standard axles, or E80s). This is the number of 80 kN single-axle loads which would cause the same damage to the pavement as the actual spectrum of axle loads.

Table 8.5: Classification of pavements and traffic for structural design purposes

PAVEMENT CLASS	BEARING CAPACITY (MILLION 80 kN AXLES/LANE)	TYPICAL TRAFFIC VOLUMES AND TYPE OF TRAFFIC	
		APPROXIMATE VEHICLES PER DAY PER LANE	DESCRIPTION
ES0,003	< 0,003	< 75	Very lightly trafficked streets, very few heavy vehicles. These roads include the transition from gravel to paved roads.
ES0,01	0,003 - 0,01		
ES0,03	0,01 - 0,03		
ES0,1	0,03 - 0,1		
ES 0,3	0,1 - 0.3	75 - 220	
ES1	0,3 - 1	220 - 700	Lightly trafficked street, mainly cars and light delivery vehicles, very few heavy vehicles.
ES3	1 - 3	>700	Medium volume, few heavy vehicles.
ES10	3 - 10	> 700	High volume and/or many heavy vehicles.
ES30	10 - 30	> 2 200	Very high volume of traffic and/or a high proportion of fully laden heavy vehicles
ES100	30 - 100	> 6 500	

For structural design, an estimate of the cumulative equivalent traffic per lane over the structural design period is required. The cumulative equivalent traffic must then fall into the pavement class for which the street is designed.

The cumulative equivalent traffic can be determined in two different ways:

- by estimation from tabulated traffic classes; and
- through detailed computation from initial and mean daily traffic axle loads, growth rates and lane-distribution factors.

The estimation of the cumulative equivalent traffic over the structural design period from tabulated values is recommended, unless more specific information is available. A detailed computation of the cumulative equivalent traffic would be applicable only when the design traffic loading is bound to be higher than 0,2 x 10^6 E80s. The designer should consider whether construction traffic is to be carried by the pavement. For Category UC and UD streets, detailed computations of traffic are normally not necessary. However, a calculation is necessary to determine an appropriate traffic loading.

For certain lightly trafficked streets, a change in the service level may be considered because of anticipated lower vehicle speeds. In these cases a lower street category should be selected (acceptance of higher risk) rather than changing the designs in the catalogue.

Computation of equivalent traffic

The detailed computation of the cumulative equivalent traffic involves

- the load equivalency of traffic;
- surveys of traffic conditions;
- projecting the traffic data over the structural design period; and
- estimating the lane distribution.

Load equivalency of traffic:
The number of E80s is termed the equivalent traffic. The load-equivalency factor relates the number of repetitions of a given axle load to the equivalent number of E80s.

This equivalency factor is a function of pavement composition, material types, definition of terminal conditions and street rideability. Table 8.6 gives average equivalency factors based on:

$$F = (P/80)^n \qquad (8.1)$$

where

n = 4,2

F = load equivalency factor

P = axle load in kN.

Pavements that are sensitive to overloading, such as shallow-structured pavements with thin cemented bases, may have n values of more than 4,2 whereas less sensitive, deep-structured pavements may have n values of less than 4,2. The designer can carry out a sensitivity analysis over the spectrum of axle loads with n values ranging from 2 to 6. This may be especially useful in the case of abnormal axle-load spectra.

The equivalent traffic can be determined by multiplying the number of axle loads (t_j) in each load group in the entire load spectrum by the relevant equivalency factor (F_j), determined from Table 8.6.

By summation the equivalent daily traffic is

$$E = \sum t_j . F_j \qquad (8.2)$$

Surveys of traffic conditions (Jordaan 1986; Haupt 1980; NITRR 1978).

The present average daily traffic is the amount of daily traffic in a single direction, averaged over the present year. This traffic can be estimated from traffic surveys carried out at some time before the initial year. Such a survey may include

- static weighing of sample of vehicles;
- dynamic weighing of all axles for a sample period (e.g. a traffic axle-weight classifier (TAWC) survey); or
- estimation procedure based on visual observation (Table 8.7 can be used to assist in this.)

<u>Projection of the traffic data over the structural design period</u>

- Projection to initial design year:

  The present average daily equivalent traffic (daily E80s) can be projected to the initial design year by multiplying by a growth factor determined from the growth rate:

$$g_x = (1 + 0{,}01.i)^x \qquad (8.3)$$

where

g = growth factor

Table 8.7: Determination of E80s per commercial vehicle

LOADING OF COMMERCIAL VEHICLES (OR TYPE OF ROAD)	NUMBER OF E80s/ COMMERCIAL VEHICLES
Mostly unladen (Category UC, residential and collector streets),	0,6
50% unladen, 50% laden (Category UA or UB, arterial roads and bus routes)	1,7
>70% laden (Category UA or UB, main arterials or major industrial routes)	2,6
Fully laden bus	3,0

Table 8.6: 80 kN single-axle equivalency factors, derived from $F = (p/80)^{4,2}$

SINGLE AXLE LOAD, P*(kN)	80 kN EQUIVALENCY FACTOR, F	SINGLE AXLE LOAD, P(kN)	80 kN AXLE EQUIVALENCY FACTOR, F
<15	0	115 - 124	5,1
15 - 24	0,004	125 - 134	7,0
25 - 34	0,019	135 - 144	9,4
35 - 44	0.062	145 - 154	12,0
45 - 54	0,150	155 - 164	16,0
55 - 64	0,320	165 - 174	20,0
65 - 74	0,590	175 - 184	26,0
75 - 84	1,000	185 - 194	32,0
85 - 94	1,600	195 - 204	39,0
95 - 104	2,400	>205	50,0
105 - 114	3,600		

*Single-axle load with dual wheels

i = growth rate (%)

x = time between determination of axle load data and opening of streets in years.

The traffic growth factor (g) is given in Table 8.8.

- Computation of cumulative equivalent traffic:

  The cumulative equivalent traffic (total E80s) over the structural design period may be calculated from the equivalent traffic in the initial design year and the growth rate for the design period. Where possible, the growth rate should be based on specific information. More than one growth rate may apply over the design period. There may also be a difference between the growth rates for total and equivalent traffic. These rates will normally vary between 2% and 10%, and a value of 6% is recommended.

  The daily equivalent traffic in the initial year is given by

  $$E_{initial} = E.\,g_x \qquad (8.4)$$

  The cumulative equivalent traffic may be calculated from

  $$N_e - E_{initial}.f_y \qquad (8.5)$$

  where

  f_y = cumulative growth factor, based on

$$f_y = \frac{365\,(1+0{,}01.i)\,[(1+0{,}01.i)^y-1]}{(0{,}01.i)} \qquad (8.6)$$

(y = structural design period).

The cumulative growth factor (f_y) is given in Table 8.9.

<u>Estimating the lane distribution of traffic</u>

On multi-lane roads, the traffic will be distributed among the lanes. Note that the distribution of total traffic and equivalent traffic will not be the same. The distribution will also change along the length of street, depending on geometric factors such as climbing or turning lanes. Suggested design factors for total traffic (B) and equivalent traffic (Be) are given in Table 8.10. As far as possible, these factors incorporate the change in lane distribution over the geometric life of a facility. The factors should be regarded as maxima and decreases may be justified.

<u>The design cumulative equivalent traffic</u>

The design cumulative equivalent traffic may be calculated by multiplying the equivalent traffic by a lane distribution factor (B_e):

$$N_e = (\Sigma t_j.F_j).g_x.f_y.B_e \qquad (8.7)$$

where

$\Sigma t_j.F_j$ = equivalent daily traffic at time of survey

g_x = growth factor to initial year (x = period from traffic survey to initial design year)

Table 8.8: Traffic growth factor (g) for calculation of future or initial traffic from present traffic

TIME BETWEEN DETERMINATION OF AXLE LOAD DATA AND OPENING OF ROAD, x (YEARS)	*g FOR TRAFFIC INCREASE, i (% PER ANNUM)								
	2	3	4	5	6	7	8	9	10
1	1,02	1,03	1,04	1,05	1,06	1,07	1,08	1,09	1,10
2	1,04	1,06	1,08	1,10	1,12	1,14	1,17	1,19	1,21
3	1,06	1,09	1,12	1,16	1,19	1,23	1,26	1,30	1,33
4	1,08	1,13	1,17	1,22	1,26	1,31	1,36	1,41	1,46
5	1,10	1,16	1,22	1,28	1,34	1,40	1,47	1,54	1,61
6	1,13	1,19	1,27	1,34	1,42	1,50	1,59	1,68	1,77
7	1,15	1,23	1,32	1,41	1,50	1,61	1,71	1,83	1,95
8	1,17	1,27	1,37	1,48	1,59	1,72	1,85	1,99	2,14
9	1,20	1,30	1,42	1,55	1,69	1,84	2,00	2,17	2,36
10	1,22	1,34	1,48	1,63	1,79	1,97	2,16	2,37	2,59

*g = $(1 + 0{,}01.i)^x$

Table 8.9: Traffic growth factor (f_y) for calculation of cumulative traffic over prediction period from initial (daily) traffic

PREDICTION PERIOD, Y (YEARS)	COMPOUND GROWTH RATE, i (% PER ANNUM) *									
	2	4	6	8	10	12	14	16	18	20
4	1 534	1 611	1 692	1 776	1 863	1 953	2 047	2 145	2 246	2 351
5	1 937	2 056	2 180	2 312	2 451	2 597	2 750	2 911	3 081	3 259
6	2 348	2 517	2 698	2 891	3 097	3 317	3 551	3 081	4 066	4 349
7	2 767	2 998	3 247	3 517	3 809	4 124	4 464	4 832	5 229	5 657
8	3 195	3 497	3 829	4 192	4 591	5 028	5 506	6 029	6 601	7 226
9	3 631	4 017	4 445	4 922	5 452	6 040	6 693	7 417	8 220	9 109
10	4 076	4 557	5 099	5 710	6 398	7 173	8 046	9 027	10 130	11 369
11	4 530	5 119	5 792	6 561	7 440	8 443	9 588	10 895	12 384	14 081
12	4 993	5 703	6 526	7 480	8 585	9 865	11 347	13 061	15 044	17 336
13	5 465	6 311	8 130	8 473	9 845	11 458	13 352	15 575	18 183	21 241
14	5 947	6 943	7 305	9 545	11 231	13 242	15 637	18 490	21 887	25 927
15	6 438	7 600	9 005	10 703	12 756	15 239	18 242	21 872	26 257	31 551
16	6 939	8 284	9 932	11 953	14 433	17 477	21 212	25 795	31 414	38 299
17	7 450	8 995	10 915	13 304	16 278	19 983	24 598	30 346	37 500	46 397
18	7 971	9 734	11 957	14 762	18 308	22 790	28 458	35 625	44 680	56 115
19	8 503	10 503	13 061	16 338	20 540	25 934	32 859	41 748	53 154	67 776
20	9 045	11 303	14 232	18 039	22 995	29 455	37 875	48 851	63 152	81 769
25	11 924	15 808	21 227	28 818	39 486	54 506	75 676	105 517	147 559	206 727
30	15 103	21 289	30 587	44 656	66 044	98 656	148 459	224 533	340 661	517 664
35	18 612	27 858	43 114	67 927	108 816	176 464	288 595	474 509	782 431	1 291 373
40	22 487	36 071	59 877	102 120	177 700	313 586	588 416	999 544	1 793 095	3 216 609

* based on $f_y = 365\ (1 + 0{,}01.i)[(1 + 0{,}01.i)^y - 1]/(0{,}01.i)$

Table 8.10: Design factors for the distribution of traffic and equivalent traffic among lanes and shoulders

TOTAL NUMBER OF TRAFFIC LANES	DESIGN DISTRIBUTION FACTOR, B_e OR B				
	SURFACED SLOW SHOULDER	LANE 1*	LANE 2	LANE 3	SURFACED FAST SHOULDER**
(a) Equivalent traffic (E80s) Factor B_e					
2	100	100	-	-	-
4	95	95	30	-	30
6	70	70	60	25	25
(b) Traffic (total axles or evu***) Factor B					
2	100	100	-	-	-
4	70	70	50	-	50
6	30	30	50	40	40

* Lane 1 is the outer or slow lane

** For multi-lane roads

*** evu = equivalent vehicle unit; one commercial vehicle = 3 evu design factors for the distribution of traffic and equivalent traffic among lanes and shoulders.

f_y = cumulative growth factor over structural design period (y = structural design period)

B_e = lane distribution factor for equivalent traffic.

To check the geometric capacity of the street, the total daily traffic towards the end of the structural design period can be calculated from

$$n = (\text{initial total daily traffic}).g_x \qquad (8.8)$$

with g_x as previously defined.

When projecting traffic over the structural design period, the designer should take into account the possibility of capacity (HRB 1985) conditions being reached, which would result in no further growth in traffic for that particular lane.

Unpaved streets

For the design of unpaved streets only the average daily traffic is required as performance is mostly a function of the total traffic, with the light : heavy split being of minor importance. This is the result of the traffic-induced deformation of properly designed unpaved streets being restricted to the upper portion of the gravel surfacing. Problems in this area are rectified during routine surface maintenance - that is, grading and spot regravelling, or by regravelling of the road.

MATERIALS

The selection of materials for pavement design is based on a combination of availability, economic factors and previous experience. These factors have to be evaluated during the design in order to select the materials best suited to the conditions.

The recommended design procedure mostly uses the standard material specification defined in Technical Recommendation for Highways *Guidelines for road construction materials* (NITRR 1984). Only abbreviated specifications are given and TRH14 should be consulted for more details. The materials are classified into various categories according to their fundamental behaviour, and into different classes according to their strength characteristics.

Description of major material types

This subsection describes the broad material types and their main characteristics. The behaviour of the different pavement types consisting of combinations of these materials is described in the following section.

Granular materials and soils (G1 to G10)

These materials show stress-dependent behaviour, and under repeated stresses deformation can occur through shear and/or densification.

A G1 is a densely-graded, unweathered, crushed-stone material compacted to 86 - 88% of apparent density. A faulty grading may be adjusted only by adding crusher sand or other stone fractions obtained from the crushing of the parent rock. G2 and G3 may be a blend of crushed stone with other fine aggregate to adjust the grading. If the fine aggregate is obtained from a source other than the parent rock, its use must be approved by the purchaser, and the supplier must furnish the purchaser with the ull particulars regarding the exact amount and nature of such fine aggregate. G4 to G10 materials range from high-quality natural gravels used in pavement layers (CBR 25-80) to lower-quality materials used in selected layers (CBR 3-15).

In unpaved streets natural gravel materials of quality G5 to G7 are recommended for the wearing course. The recommended plasticity index limits for G4 or G5 will not apply if the material is used as a gravel wearing course. Maximum particle size limits also have to be reduced for use as a gravel wearing course.

Asphalt hot-mix materials (BC to TS)

Asphalt hot-mix materials are visco-elastic and under repeated stresses they may either crack or deform or both. Normally a continuously graded bitumen hot-mix (BC) will have a higher stability and lower fatigue life than a semi-gap-graded (BS) material. Tar hot-mixes (TC, TS) will normally have lower fatigue lives than the equivalent bitumen hot-mixes. Usually the stability of a tar mix is the same as or higher than that of the equivalent bitumen mix. Tar mixes have seldom been used in South Africa due to low durability.

Bituminous cold-mix materials

Natural gravel or recycled material may be treated with emulsion (ETM) or foamed bitumen (FTM) to increase the stiffness and strength, after an initial curing period (SABITA 1993; Csanyi 1960). The residual binder content for foam- and emulsion-treated material is normally below 5% by mass. A distinction is made between emulsion stabilisation (GEMS) with residual binder contents between 1,5 and 5% by mass and emulsion modification (ETBs) with residual binder contents between 0,6 and 1,5% by mass.

Foamed bitumen and emulsion treatment may be used on new construction projects to treat the locally available material, enabling the use of this material in the pavement base layer or on rehabilitation projects by treating the existing base material. Emulsion-treated material may be mixed on site by conventional construction equipment, deep-milling machines or plant. The emulsion-treated material should however, be placed and compacted immediately. Foam-treated material may be mixed in a plant or on site by using deep-milling machines, and may be stockpiled for periods up to three months (KZN 1997). A further advantage of using these materials on rehabilitation projects is that the streets may be opened to traffic very soon after construction. The unsurfaced base layer may even be exposed to traffic for limited periods of time without material loss.

Portland cement concrete (PCC)

Concrete is an elastic, brittle material possessing low tensile strength and it may thus crack under excessive repeated flexure. In this document only one concrete strength is considered.

Cemented materials (C1 to C4)

As with concrete, cemented materials are elastic, possess low tensile strength and may crack under repeated flexure. These materials also crack because of shrinkage and drying. By applying an upper limit to the strength specification, wide shrinkage cracks can be avoided. Because of the excessive shrinkage cracking of C1 materials, they are not generally used.

A C2 material will be used when a non-pumping erosion-resistant layer is required (as for the subbase under a concrete pavement)

C3 and C4 materials can be used as replacements for granular layers in bases and subbases. They can be treated with cement, lime, slagment, lime/flyash mixtures or various combinations of pozzalanic binders, depending on the properties of the natural materials.

Surfacing (AG to AO; S1 to S8)

The surfacings range from high-quality asphalt surfacings to surface treatments, surface maintenance measures such as rejuvenators, and diluted emulsion treatments.

Macadams (WM to PM)

These traditional, high-quality, but also labour-intensive, pavement materials can be used in the place of GI to G4 materials. However, specific knowledge of construction techniques is required. These materials are less affected by water than the usual granular materials and their use should be considered, especially in wet regions.

Paving blocks (S-A, S-B)

The use of interlocking concrete paving blocks (S-A) is limited to low-speed (<50 km/h) streets or terminal areas. Blocks with good interlocking shapes should be used.

The use of clay bricks as paving blocks (S-B) should be limited to Category UC and UD streets. The durability of clay bricks is expected to be poorer than that of concrete paving blocks.

Cast in-situ blocks

A South African developed patented welded plastic geocell has been successfully used in pavements for local access streets, and results in a flexible portland cement concrete pavement. These cells form a square with a side length of 150mm. After tensioning the plastic, which acts as a formwork, alternative methods of placing portland cement concrete can be used. The one is to provide premixed or ready-mix concrete. A more suitable labour-intensive method is to place the coarse aggregate in the cells and then apply compaction, and a cement grout is used to fill the voids. The process is a cement-grouted variant of the well-known waterbound macadam. The plastic cells limit the amount of shrinkage of blocks formed in situ. By first placing the coarse aggregate, only 40% of the volume of the slab, which is the typical void ratio in a single-sized aggregate, has to be mixed and handled (Visser 1994; Visser and Hall 1999).

PAVEMENT TYPES

For paved roads there are five major pavement types, namely granular, bituminous, concrete and cemented-base pavements and pavements with paving blocks. Unpaved roads constitute a separate pavement type.

Behaviour of different pavement types

Paved streets

The behaviour of a pavement and the type of distress that will become the most critical vary with the type of pavement. The behaviour of these different pavement types will determine the type of maintenance normally required and may also influence the selection of the pavement type. A brief description of the typical behaviour of each pavement type is given below.

<u>Untreated granular-based pavements</u>

This type of pavement comprises a thin bituminous surfacing, a base of untreated gravel or crushed stone, a granular or cemented subbase and a subgrade of various soils or gravels. The mode of distress in a pavement with an untreated subbase is usually deformation, arising from shear or densification in the untreated materials. The deformation may manifest itself as rutting or as longitudinal roughness eventually leading to cracking. This is illustrated in Figure 8.3(a).

In pavements with cemented sub-bases, the subbase improves the load-carrying capacity of the pavement, but at some stage the subbase will crack under traffic. The cracking may propagate until the layer eventually exhibits properties similar to those of a natural granular material. It is unlikely that cracking will reflect to the surface, and there is likely to be little rutting or longitudinal deformation until after the subbase has cracked extensively. However, if the subbase exhibits large shrinkage or thermal cracks, they may reflect to the surface.

Recent work has shown that the post-cracked phase of a cement-treated subbase under granular and bituminous bases adds substantially to the useful life of the pavement. Deflection measurements at various depths within the pavement have indicated that the initial effective modulus of this material is high - 3 000 to 5 000 MPa as shown in Figure 8.3(c).

This relatively rigid subbase generally fatigues under traffic, or in some cases even under construction traffic, and assumes a lower effective modulus (800 to 1 000 MPa). This change in modulus does not normally result in a marked increase in deformation, but the resilient deflection and radius of curvature do change, as shown in Figure 8.3(d).

In the mechanistic design (Freeme, Maree and Viljoen 1982) these phases have been termed the pre-cracked and post-cracked phases. The design accommodates the changes in modulus of the subbase and, although the safety factor in the base will be reduced, it will still be well within acceptable limits.

The eventual modulus of the cemented subbases will depend on the quality of the material originally stabilised, the cementing agent, the effectiveness of the mixing process, the absolute density achieved, the durability of the stabilisation and the degree of cracking. The ingress of moisture can affect the modulus in the post-cracked phase significantly. In some cases the layer may behave like a good-quality granular material with a modulus of 200 to 500 MPa, but in other cases the modulus may be between 50 and 200 MPa. This change is shown diagrammatically in Figure 8.3(c).

The result is that the modulus of the cemented subbase assumes very low values and this causes fatigue and high shear stresses in the base. Generally, surface cracking will occur and, with the ingress of water, there may be pumping from the subbase.

For high-quality, heavily trafficked pavements it is necessary to avoid materials that will eventually deteriorate to a very low modulus. Many of these lower-class materials have, however, proved to be adequate for lower classes of traffic.

The surfacing may crack owing either to hardening of the binder as it ages or to load-associated fatigue cracking. The strength of granular materials is often susceptible to water, and excessive deformation may occur when water enters through surface cracks. The water-susceptibility of a material depends on factors such as grading, the PI of the fines, and density. Waterbound macadams are less susceptible to water than crushed-stone bases and are therefore preferred in wet regions.

<u>Bituminous pavements</u>

These pavements have a bituminous layer more than 80 mm thick. They can be subdivided into two major groups, namely bitumen- and tar-base pavements:

Bitumen-base pavements:
In bitumen-base pavements both deformation and fatigue cracking are possible. Two types of subbase are recommended, namely either an untreated granular subbase or a weakly stabilised cemented subbase. Rutting may originate in either the bituminous or the untreated layers, or in both. This is illustrated in Figure 8.3(b). If the subbase is cemented there is a probability that shrinkage or thermal cracking will reflect through the base to the surfacing, especially if the bituminous layer is less than 150 mm thick or if the subbase is excessively stabilised. Maintenance usually consists of a surface treatment to provide better skid resistance and to seal small cracks, an asphalt overlay in cases where riding quality needs to be restored and when it is necessary to prolong the fatigue life of the base, or recycling of the base when further overlays are no longer adequate.

Tar-base pavements:
The fatigue life of a tar premix is well below that of most asphalt hot-mix materials. Only weakly cemented subbases are used. The main distress appears to be cracking of the cemented subbase, followed by fatigue of the tar base.

Maintenance for tar bases is the same as for bituminous bases.

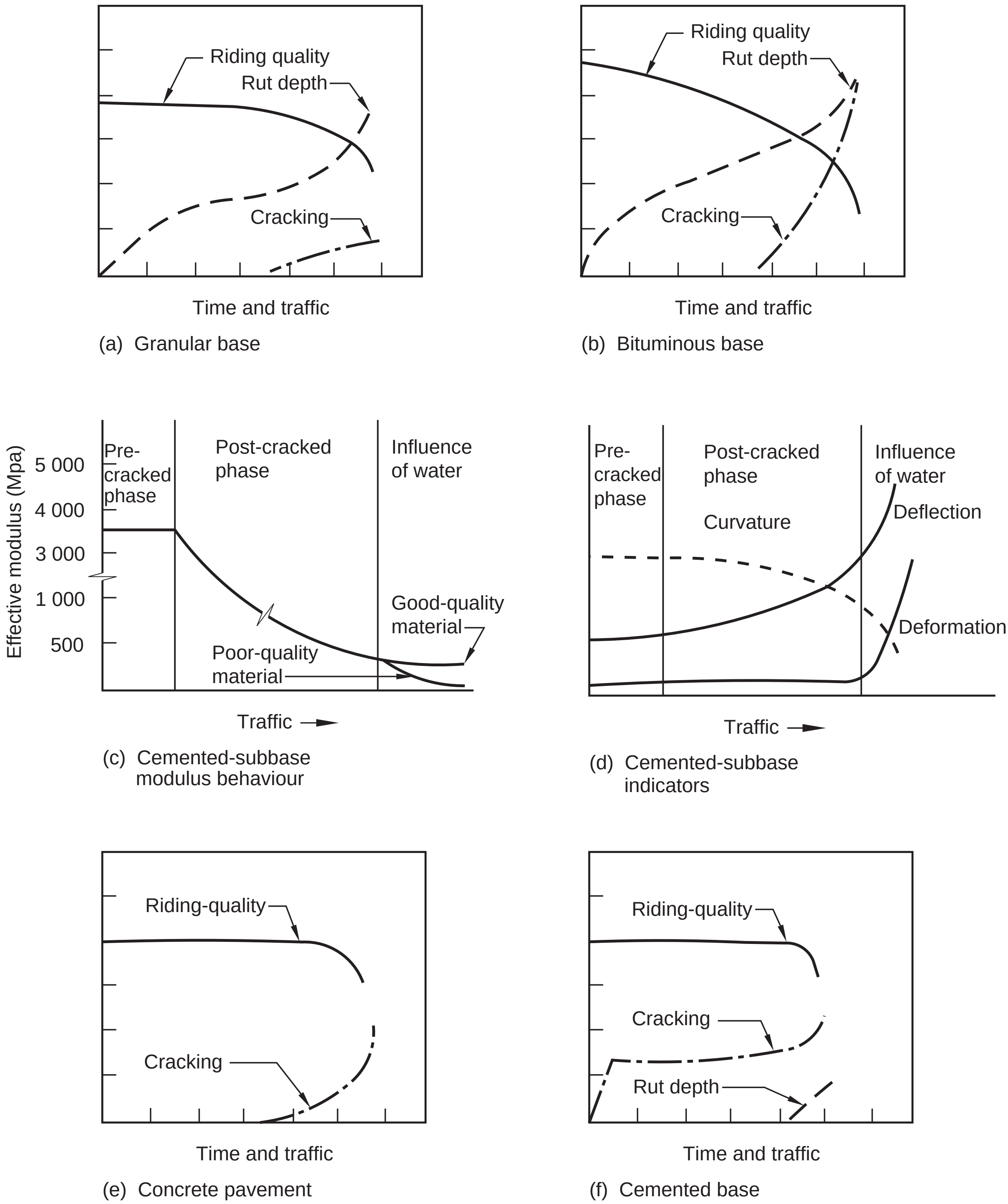


Figure 8.3: General pavement behaviour characteristics

<u>Bituminous cold-mix based pavements</u>

Although the binders in emulsion and foam-treated materials are viscous, the material is stiff and brittle much like a cement-treated material after curing. The initial field stiffness values of these materials may vary between 800 and 2 000 Mpa, depending on the binder content and parent material quality. These values will gradually decrease with increasing traffic loading to values typical for granular materials between 150 and 500 MPa. Early indications are that this reduction is caused by the breakdown (effective fatigue) of the bonded layer into particle sizes smaller than the thickness of the layer (Theyse 1997).

Field performance of pavements with emulsion- and foam- treated base layers indicates that they are not as sensitive to overloading as a pavement with a cement- treated base, and do not pump fines from the subbase.

Concrete pavements

In concrete pavements, most of the traffic loading is carried by the concrete slab and little stress is transferred to the subgrade. The cemented subbase provides a uniform foundation and limits pumping of subbase and subgrade fines. Through the use of tied shoulders, most of the distress stemming from the edge of the pavement can be eliminated and slab thickness can also be reduced. Distress of the pavement usually appears first as spalling near the joints, and then may progress to cracking in the wheel paths. Once distress becomes evident, deterioration is usually rapid. See Figure 8.3(e).

Maintenance consists of patching, joint repair, crack repair, under-sealing, grinding, or thin concrete or bituminous overlays. In cases of severe distress, thick concrete, bituminous or granular overlays will be used, or the concrete may be recycled.

Cemented-base pavements

In these pavements, most of the traffic stresses are absorbed by the cemented layers and a little by the subgrade. It is likely that some block cracking will be evident very early in the life of the cemented bases; this is caused by the mechanism of drying shrinkage and by thermal stresses in the cemented layers. Traffic-induced cracking will cause the blocks to break up into smaller ones. These cracks propagate through the surfacing. The ingress of water through the surface cracks may cause the blocks to rock under traffic, resulting in the pumping of fines from the lower layers. Rutting or roughness will generally be low up to this stage but is likely to accelerate as the extent of the cracking increases. See Figure 8.3(f).

Pavements consisting of cemented bases on granular subbases are very sensitive to overloading and to ingress of moisture through the cracks. When both the base and the subbase are cemented, the pavement will be less sensitive to overloading and moisture. The latter type of pavement is generally used.

The shrinkage cracks that form early in the life of the pavement may be rehabilitated by sealing. Once traffic-load-associated cracking has become extensive, rehabilitation involves either the reprocessing of the base, or the application of a substantial bituminous or granular overlay.

Paving blocks

Many types of interlocking and non-interlocking segmental blocks are used in a wide variety of applications, which range from footpaths to driveways to heavily loaded industrial stacking and servicing yards. The use of segmental block pavements is a relatively recent phenomenon in South Africa. The popularity of these blocks is increasing due to a number of factors:

- the blocks are manufactured from local materials;
- they can either provide a labour-intensive operation or can be manufactured and laid by machine;
- they are aesthetically acceptable in a wide rage of applications; and
- they are versatile as they have some of the advantages of both flexible and concrete pavements.

In current practice a small plate vibrator is used to bed the blocks into a sand bedding of approximately 20 mm and to compact jointing sand between individual blocks. The selection of the right type of sand for these purposes is important, since a non-plastic material serves best as bedding while some plastic content is required to fill the joints.

Properly laid block pavements are adequately waterproofed and ingress of large quantities of water into foundations does not occur. The procedures for the structural design of segmental block pavements are presented in UTG2 *Structural design of segmental block pavements in southern Africa* (NITRR 1984) and are applicable to both industrial and street uses.

Segmental blocks are manufactured with vertical square side faces. Those that interlock are shaped so as to allow them to fit “jigsaw” fashion into a paved area. They can be made of pressed concrete, fired-clay brick or any other material. The current recommended minimum strength for structural use is given as a wet compressive strength of not less than 25 MPa.

Block pavements require the paved area to be “contained” either by kerbs or by other means of stopping lateral spread of the block. This is a requirement for both interlocking and non-interlocking shapes. Lateral movements are induced by trafficking and these movements cause breaks in the jointing sand. The associated opening-up of the block pavement makes it more susceptible to the ingress of surface water. In heavily loaded areas interlocking shapes have advantages over non-interlocking shapes, especially if vehicles with a slewing action are involved.

Experience has shown that joints should be 2 to 5 mm wide. Geometric design should follow practices for other pavements. Variable street widths, curves and junctions do not present problems in practice, since the blocks are small and can easily be cut and placed to suit the geometry of the pavement. In practice, the minimum cross-fall for block pavements should be one per cent. For wide areas of industrial paving, special care should be taken to ensure that the cross-fall of the surface is adequate. Cambered cross-sections are also satisfactory.

The joints between the blocks seal better with time due to the action of weathering and the addition of street detritus to the joints, thereby improving the total strength of the block pavement. One per cent falls (minimum) to the surface of block pavements allow water to drain across the pavement, reducing ingress by absorption through the joints, and eliminate ponding. The joints between the blocks on steep gradients may form the drainage paths for rainwater. In such cases the pattern of the blocks is an important consideration. Experience has shown that a herringbone pattern is best for use on steep gradients and for industrial paving.

An advantage of the blocks is that they can be re-used. They can be lifted if repairs have to be carried out to failed areas of subbase or if services have to be installed and can be relaid afterwards. As far as the design of segmental block pavements is concerned, this re-use of the blocks has no disadvantages.

Little maintenance work is required with segmental block paving. Maintenance involves the treatment of weeds and the correcting of surface levels if the initial construction had been poor. The correction of surface levels is done by removing the area of blocks affected, levelling the subbase, compacting the subbase (often with hand hammers) and replacing the blocks.

Segmental paving provides an exciting addition to the pavement construction methods possible in southern Africa.

Unpaved streets

The most common causes of poor performance of gravel streets are slipperiness and potholing when wet, and excessive dust and ravelling when dry. The formation of corrugations is normally the result of inadequate compaction or low cohesion combined with traffic. Frequent maintenance (e.g. grading, watering and the addition of material) is therefore necessary.

ENVIRONMENT

The environment is characterised by topography, the climatic conditions (moisture and temperature) under which the street will function, and the underlying subgrade conditions. Environmental factors must be taken into account in the design of pavement structures.

Topography

Topography is dealt with in Chapter 6: *Stormwater Management.* The importance of its influence on the structural design and functional use of streets is clearly reflected in the drainage and maintenance requirements of streets in general. Macro drainage is relevant to this discussion. In rolling and mountainous terrain there may be steep gradients which result in the erosion of gravel streets and, in particular, erosion of their drainage facilities, with direct implications for their safety and functional use. Streets that cross contours at an angle, or even perpendicularly, pose the most drainage problems. In such cases functional rather than structural requirements may demand that a street be paved or protected from erosion. It is therefore important that requirements described in the chapters on Layout Planning and Stormwater Management be met before one embarks on the structural design.

Unpaved streets

Streets with high longitudinal gradients are more likely to occur in mountainous areas or even hilly terrain. A 6% longitudinal street gradient is an average value above which erosion problems may occur, and slopes steeper than this would warrant additional attention. The upper range of suggested PI could effectively counteract erosion, but may result in unacceptable slipperiness on steep slopes. Local conditions should be considered in the detailed evaluation.

Climate and structural design

The climate will largely determine the rate of weathering of natural rock and its products, the durability of weathered natural street-building materials and - depending on drainage conditions - the stability of untreated materials in the pavement. The climate may also influence the equilibrium moisture content within the pavement layers. The designer should always consider climatic conditions and avoid using materials that are excessively water-susceptible or temperature-sensitive in adverse conditions. It is also possible to accommodate climatic conditions by either adjusting California Bearing Ratio (CBR) values or by weighting the equivalent traffic (not both).

Chapter 8

Roads: Materials and construction

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

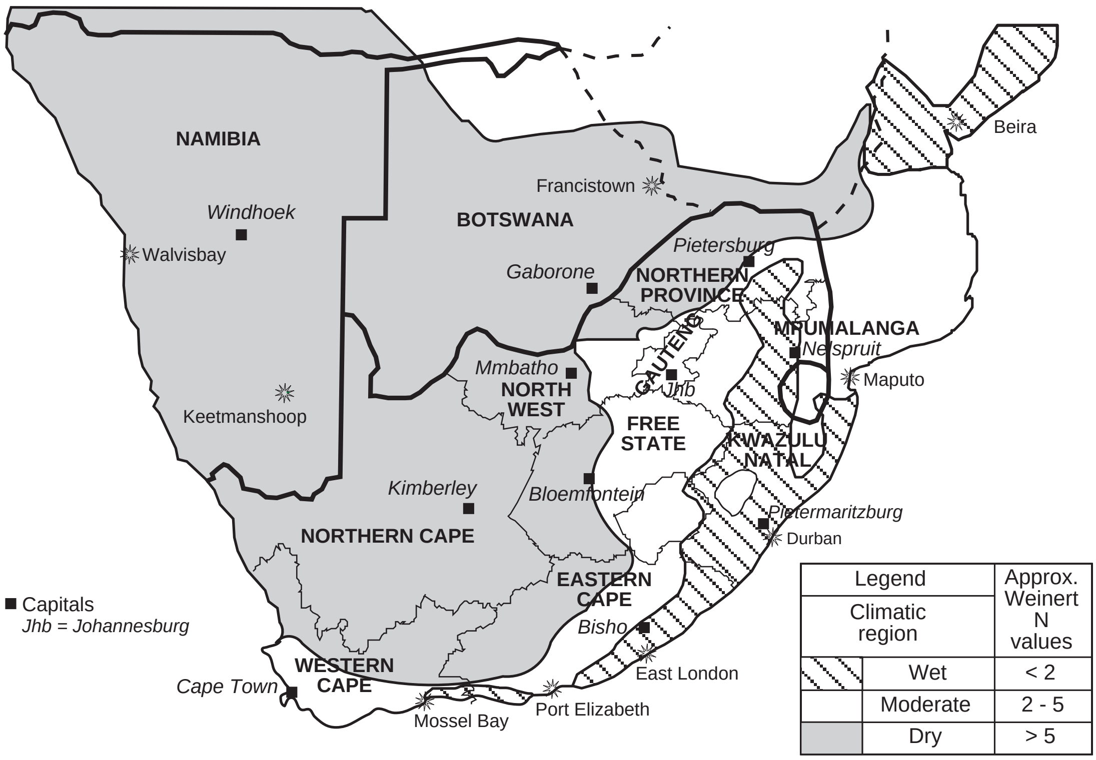


Figure 8.4: Macro-climatic regions of southern Africa

Southern Africa can be divided into three climatic regions:

- a large dry region;
- a moderate region; and
- a small wet region.

Figure 8.4 is a map of southern Africa, which indicates the different climatic regions. These are macroclimates and it should be kept in mind that different microclimates may occur within these regions. This is particularly important where such local microclimates have a high moisture content. This will have a direct influence on moisture-susceptible materials in basic access streets which require specific drainage considerations.

Climate and subgrade California Bearing Ratio (CBR)

The design parameter for the subgrade is the soaked CBR at a representative density. For structural design purposes, when a material is classified according to the CBR, it is implied that not more than 10% of the measured values for such a material will fall below the classification value. A proper preliminary soil survey should be conducted.

It is current practice to use soaked CBR values, but using them in dry regions may be over-conservative (Jordaan 1986). It is suggested that the CBR value of a material be increased if the in situ conditions are expected to be unsoaked, e.g. in dry regions (Haupt 1980). An estimate of the CBR at the expected moisture conditions, i.e. at optimum moisture content (OMC) or, say, 75 per cent of OMC, can be determined in the laboratory by refraining from soaking the samples before CBR testing or even drying back to the required moisture content.

The dynamic cone penetrometer (DCP) can be used to determine the in-situ CBR and variations in in-situ strengths (Jordaan 1986). The in-situ CBRs determined with the DCP can be calibrated by doing laboratory-soaked CBRs. If material parameters such as grading modulus (GM), plastic limit (PL) and dry density (DD) are included in the analysis, typical relations can be used to derive CBR values (Sampson 1984). The relevant equation is:

$$\log_e \text{CBR} = 1{,}1\ (\log_e \text{DCP}) + 0{,}85\ (\text{GM}) - 0{,}031\ (\text{PL}) - 0{,}001\ (\text{DD}) + 7{,}4 \qquad (8.9)$$

where loge is the natural logarithm.

For the material types under consideration, the CBR is determined at a 2,54 mm depth of penetration, with DCP penetration in millimetres per blow.

It is current practice for the design parameter for subgrade to be the soaked California Bearing Ratio (CBR) for paved streets. It is recommended that unsoaked (field) CBR values should be used particularly in dry regions (Haupt 1980; Emery 1984 and 1987). The dynamic cone penetrometer (DCP) is the ideal instrument for such an approach (Kleyn and van Zyl 1987; Kleyn 1982). However, it should be pointed out that soaked CBR values may be required for wet regions where no proper drainage can be provided or for wet-season passability on unpaved streets. When the DCP is used, care should be taken that the moisture content is a fair representation of the moisture content over long periods of time.

The requirement for subgrade or fill CBR is a soaked CBR of at least 3 at 90% Mod AASHTO density in wet areas, and an in-situ CBR of 3 in dry and moderate areas. The material should also have a maximum swell of 1,5% at 100% Mod AASHTO compaction to ensure that it is not too expansive. If the CBR values are determined in the field with the DCP, the subgrade areas with a field CBR of less than 3 will need special treatment. If the field CBR values are in excess of 45 over a depth of at least 150mm at a density of 95% Mod AASHTO, the subgrade can be considered to be subbase quality, and only a base would be needed.

Material depth

The term "material depth" is used to denote the depth below the finished level of the street to which soil characteristics have a significant effect on pavement behaviour. Below this depth, the strength and density of the soils are assumed to have a negligible effect on the pavement. The depth approximates the cover for a soil with CBR of 1 - 2. However, in certain special cases this depth may be insufficient. These cases are listed in the section dealing with practical considerations (subgrade below material depth).

Table 8.11 specifies the materail depth used for determing the design CBR of the subgrade for different street categories.

Table 8.11: Material depths to be used for determining the design CBR of the subgrades

ROAD CATEGORY	MATERIAL DEPTH (mm)
UA	1 000
UB	800
UC	600
UD	400

Delineation of subgrade areas

Any street development should be subdivided into significant subgrade areas. However, if the delineation is too fine it could lead to confusion during construction. The preliminary soil survey should delineate subgrade design units on the basis of geology, pedology, topography and drainage conditions - or major soil boundaries - on site so that an appropriate design CBR for each unit can be defined.

The designer should distinguish between very localised good or poor soils and more general subgrade areas. Localised soils should be treated separately from the rest of the pavement factors. Normally, localised poor soils will be removed and replaced by suitable material.

Design CBR of subgrade

For construction purposes the design subgrade CBR is limited to four groups, as shown in Table 8.12.

Table 8.12: Subgrade CBR groups used for structural design

CLASS	SUBGRADE CBR
SG1	>15
SG2	7 to 15
SG3	3 to 7
SG4	<3*

* Special treatment required.

The CBR is normally determined after samples have been soaked for four days. Special measures are necessary if a material with a CBR of less than 3 is encountered within the material depth. These measures include stabilisation (chemical or mechanical), modification (chemical), or the removal or addition of extra cover. After the material has been treated, it will be classified under one of the remaining three subgrade groups.

Design CBR on fill

When the street is on fill, the designer must avail himself of the best information available on the local materials that are likely to be used. The material should be controlled to at least the material depth. TRH10 (NITRR 1984) should be consulted when a material with a CBR of less than 3 is used in the fill.

Design CBR in cut

The design CBR of the subgrade in a cut should be the 10 percentile CBR encountered within the material depth.

STRUCTURAL DESIGN METHODS

Design methods for paved streets

There are a number of design methods of varying complexity at the disposal of the designer. Some of these are purely empirical and others incorporate some measure of rationality, and were developed both locally and abroad.

The designer must always bear the limitations of a particular design method in mind. Most of the purely empirical design methods were developed from data where the design bearing capacity did not exceed 10 to 12 million standard axles. The purely empirical design methods are also limited in their application to conditions similar to those for which they were developed. The designer must therefore make a critical assessment of the applicability of the design method to his design problem. Locally developed methods should then also have an advantage in this regard.

It must also be kept in mind that, although these design methods will predict a certain bearing capacity for a pavement structure, there are many factors that will influence the actual bearing capacity of the pavement, and the predicted value should be regarded only as an estimate. It is therefore better to apply various design methods, with each method predicting a somewhat different bearing capacity. This will assist the designer to develop a feeling for the range of bearing capacity for the pavement, rather than stake everything on a single value.

The "Catalogue" design method

This document focuses mainly on the use of the catalogue of pavement designs (CSRA 1996; CUTA 1987; Hefer 1997; Theyse 1997) included in Appendix A. However, this does not exclude the use of any of the other proven design methods. Most of the pavement designs in the catalogue were developed from mechanistic-empirical design, although some are based on the DCP design method and others are included on the basis of their field performance.

The catalogue approach is a fairly straightforward pen-and-paper method and does not require access to a computer.

The California Bearing Ratio (CBR) cover design method

The California Bearing Ratio (CBR) design method was developed in the 1950s from empirical data (Yoder and Witczak 1975). The method is based on the approach of protecting the subgrade by providing enough cover of sufficient strength to protect the subgrade from the traffic loading. CBR-cover design charts were developed for different subgrade CBR strengths and traffic loadings. The applicability of this method should be evaluated critically before it is applied to local environmental and traffic conditions.

This method is a pen-and-paper method and no access to a computer is required.

The AASHTO Guide for Design of Pavement Structures

The AASHTO Guide for Design of Pavement Structures provides the designer with a comprehensive set of procedures for new and rehabilitation design and provides a good background to pavement design (AASHTO 1993). The design procedures in the guideline document are, however, empirical, and were mostly developed from the results of the AASHTO Road Test carried out in the late 1950s and early 1960s.

Although some software based on the procedures in the AASHTO design guide is commercially available, the procedure may be applied just as well by hand.

The Dynamic Cone Penetrometer (DCP) method

The DCP design method was developed locally during the 1970s. The original method was based on the CBR-cover design approach and later correlated with heavy vehicle simulator (HVS) test results. This method incorporates the concept of a balanced pavement structure in the design procedure (Kleyn and van Zyl 1987). If used properly, designs generated by this method should have a well-balanced strength profile with depth, meaning that there will be a smooth decrease in material strength with depth. Such balanced pavements are normally not very sensitive to overloading. Some knowledge of typical DCP penetration rates for road-building material is required to apply this method.

DCP design may be done by hand, but if DCP data need to be analysed, access to a computer and appropriate software is necessary.

South African Mechanistic Design Method

The South African Mechanistic Design Method (SAMDM) (van Vuuren et al 1974; Walker et al 1977; Paterson and Maree 1978; Theyse et al 1996) was developed locally and is one of the most comprehensive mechanistic-empirical design methods in the world (Freeme, Maree and Viljoen 1982). This method may be used very effectively for new and rehabilitation design. Some knowledge of the elastic properties of materials as used by the method is required, and experience in this regard is recommended. In the case of rehabilitation design or upgrading, field tests such as the DCP and Falling Weight Deflectometer (FWD) may be used to determine the input parameters for the existing structure.

Access to a computer and appropriate software is essential for effective use of this method, as well as for analysing DCP and FWD data.

Design methods for unpaved streets

Unlike sealed roads, where the application of a bituminous surfacing results in a semi-permanent structure (for up to 20 years) in which deformation or failure is costly to repair and usually politically unacceptable, unsealed roads are far more forgiving. Routine maintenance is essential and localised problems are rectified relatively easily. For this reason, the design process for unsealed roads has never progressed to the sophisticated techniques developed for sealed roads.

The main principles in designing unsealed roads are

- to prevent excessive subgrade strain; and
- to provide an all-weather, dust-free surface with acceptable riding quality.

These two requirements are achieved by providing an adequate thickness of suitable material, constructed to a suitable quality. A simple design technique covering thickness and materials has been developed for South Africa and is summarised in TRH14 *Guidelines for road construction materials* (NITRR 1984c).

PRACTICAL CONSIDERATIONS

Surface drainage

Experience has shown that inadequate drainage is probably responsible for more pavement distress in southern Africa than inadequate structural or material design. Effective drainage is essential for good pavement performance, and it is assumed in the structural design procedure.

Drainage for basic access streets

Effective drainage is a prerequisite in the structural design of basic access streets. Drainage design is integral in stormwater management. As outlined in the principles of stormwater management, the design should allow for non-structural and structural measures to cope with minor and major storms. The non-structural measures are related to optimising the street layout and the topography to retard stormwater flow and curb the possible associated damage. Structural measures include not only the provision of culverts, pipes or channels, but also the street itself. With minor storms, structural measures should ensure that water is shed from the street into side drainage channels, and with major storms, these measures should limit the period of impassability while the street functions as a drainage channel itself. In the latter case the street should be paved.

In addition to paving basic access streets for reasons of drainage, erosion control and wet-weather accessibility, other factors such as dust and social issues may play a role.

Included in the social issues are politics, adjacent schools and hospitals, and the use of the streets as public areas.

Unpaved basic access streets therefore require side drainage channels that are lower than the street level to ensure that water is drained off the street into the side channel. These side channels should be carried through the main street at intersections. A maximum cross-fall of 5% is suggested for the street. For paved streets this cross-fall can be reduced to 3%. In Figure 8.5 typical cross-sections of basic access streets are illustrated. For basic access streets, excess water can be handled in side channels or even on the street itself, acting as a channel-and-street combination, and be led to open areas (e.g. sports fields, parks) for dissipation. For an unpaved network, channels - as shown in Figure 8.6 - and not pipes are required. Pipes can be considered only where all the basic access streets are surfaced owing to the problem of silting, or where the gradient of the pipe and design of inlets and outlets are such that silting and blocking will not occur (NITRR, 1984a).

Channels should be designed to prevent silting or ponding of stagnant water, yet avoid excessive erosion. Ponding is of particular importance as it can result in water soaking into the structural layers of the pavement. The draft TRH17 (NITRR, 1984b) gives guidelines on the design of open channels to prevent silting.

The longitudinal gradient of a channel and the material used determine the amount of scouring or erosion of such channels. Table 8.13 provides the scour velocities for various materials and guidance on the need to line or pave channels. Linings of hand-packed stone can be as functional as concrete linings. As a rough guide it is suggested that an unpaved channel should not be steeper than 2% (1:50).

Accesses to dwelling units should provide a smooth entry, whilst preventing stormwater in the street or channel from running onto properties. Where side drainage is provided to streets, special attention should be paid to the design of access ramps, or the elimination of the need for ramps to safeguard the functioning of the drainage channels.

Kerbing is not used extensively on basic access streets if they are surfaced, but edging may be used as an alternative when the shoulder or sidewalk material is of inadequate stability. However, it should be stressed that the shoulder of a surfaced basic access street should be constructed with

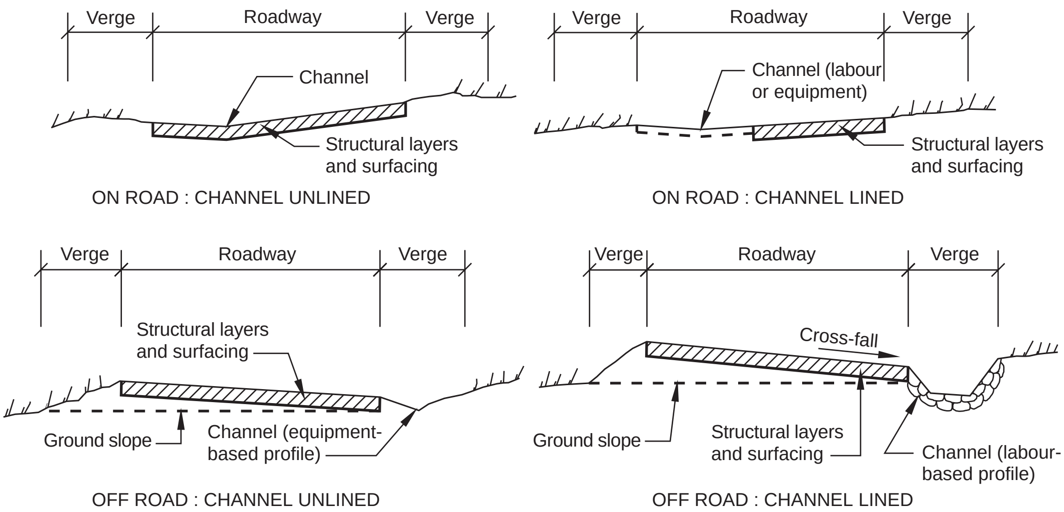


Figure 8.5: Typical basic access street cross-sections

material of at least the same quality as the subbase (Netterberg and Paige-Green 1988).

The shoulder should preferably be protected with a bituminous surfacing. The cost of this can be high however, and the decision will have to be based on affordability.

Erosion control for tertiary ways

Erosion control is considered to be the main criterion in the design of tertiary ways. Stormwater must be accommodated by ditches and drains on the sides of the tertiary ways. In Figure 8.6 typical detail is given of such ditches. Detail is also given of stilling ponds, catchwater drains and check dams. These should be seen as typical examples illustrating the principles involved. Check dams are used on downhill tertiary ways to dissipate the energy of the stormwater and to form natural steps.

Table 8.13: Scour velocities for various materials

MATERIAL	ALLOWABLE VELOCITY(m/s)
Fine sand	0,6
Loam	0,9
Clay	1,2
Gravel	1,5
Soft shale	1,8
Hard shale	2,4
Hard rock	4,5

When low points are reached, drifts and dished drains can be used to give preference to the flow of water without major structural requirements. Erosion protection on the approaches must be provided for. Details of typical drifts and dish drains are shown in Figures 8.7 and 8.8.

Tertiary ways would normally be constructed from the in situ material. The use of vegetation to prevent erosion is highly recommended and can be achieved by various means. Grass-blocks are but one example where vegetation is used to prevent erosion (Figure 8.9). These should, however, be regularly maintained to avoid a build-up of grass and silt.

Subsurface drainage

Subsurface drainage design is a specialised subject and both the infiltration of surface water and the control of subsurface water have to be considered. The basic philosophy is to provide effective drainage to (at least) material depth so that the pavement structure does not become excessively wet.

Subsurface drainage problems can be reduced most effectively by raising the road above natural ground level. This cannot always be done in the urban situation and the provision of side drains adjacent to the road, to a depth as low as possible beneath the road surface is equally effective.

If neither of these two options is practical, and ground water or seepage flows prevail, some form of cut-off trench with an interceptor drain may be necessary. These are expensive and other options (such as a permeable layer beneath the subbase) could also be considered.

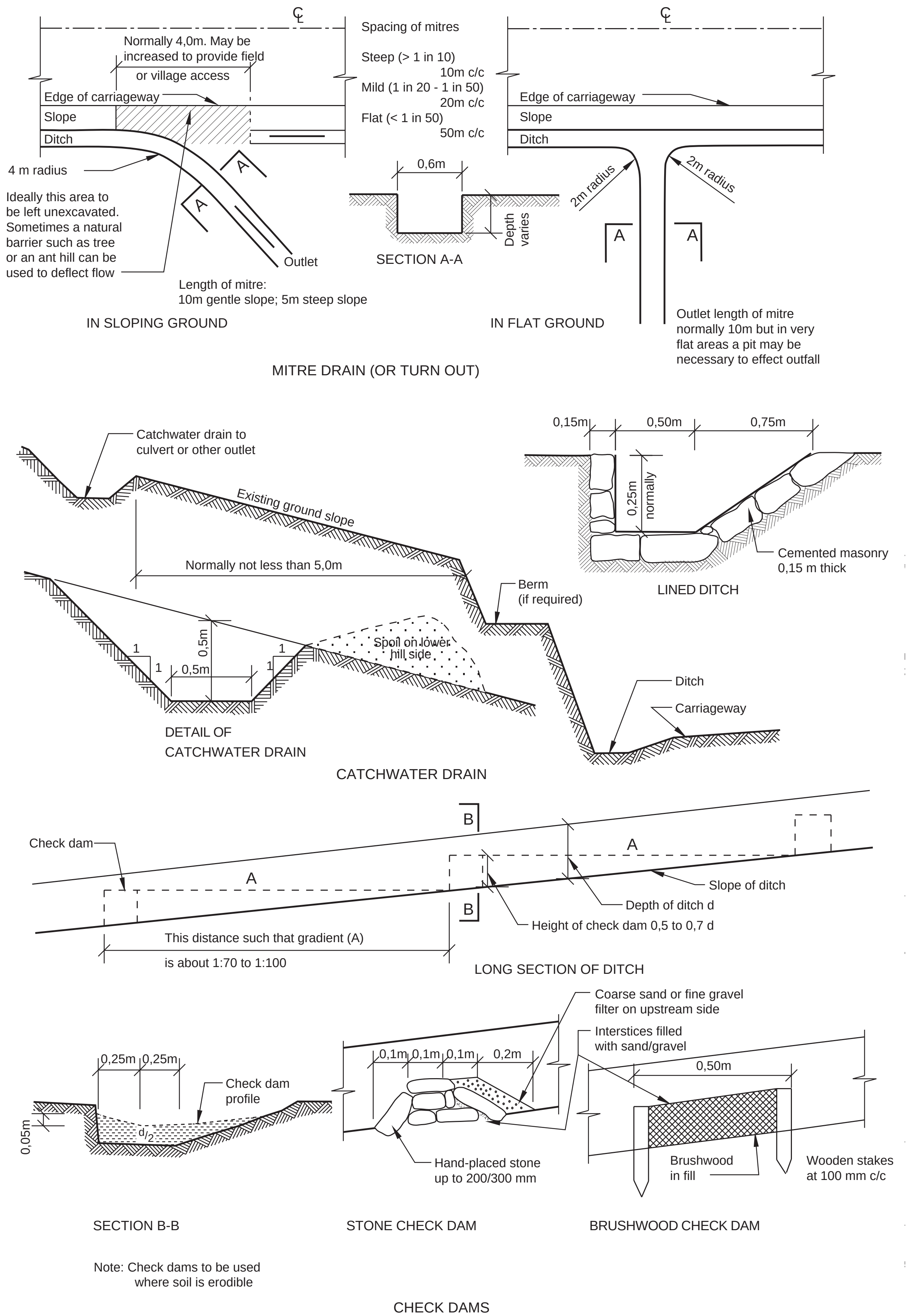


Figure 8.6: Tertiary ways: ditches and drains

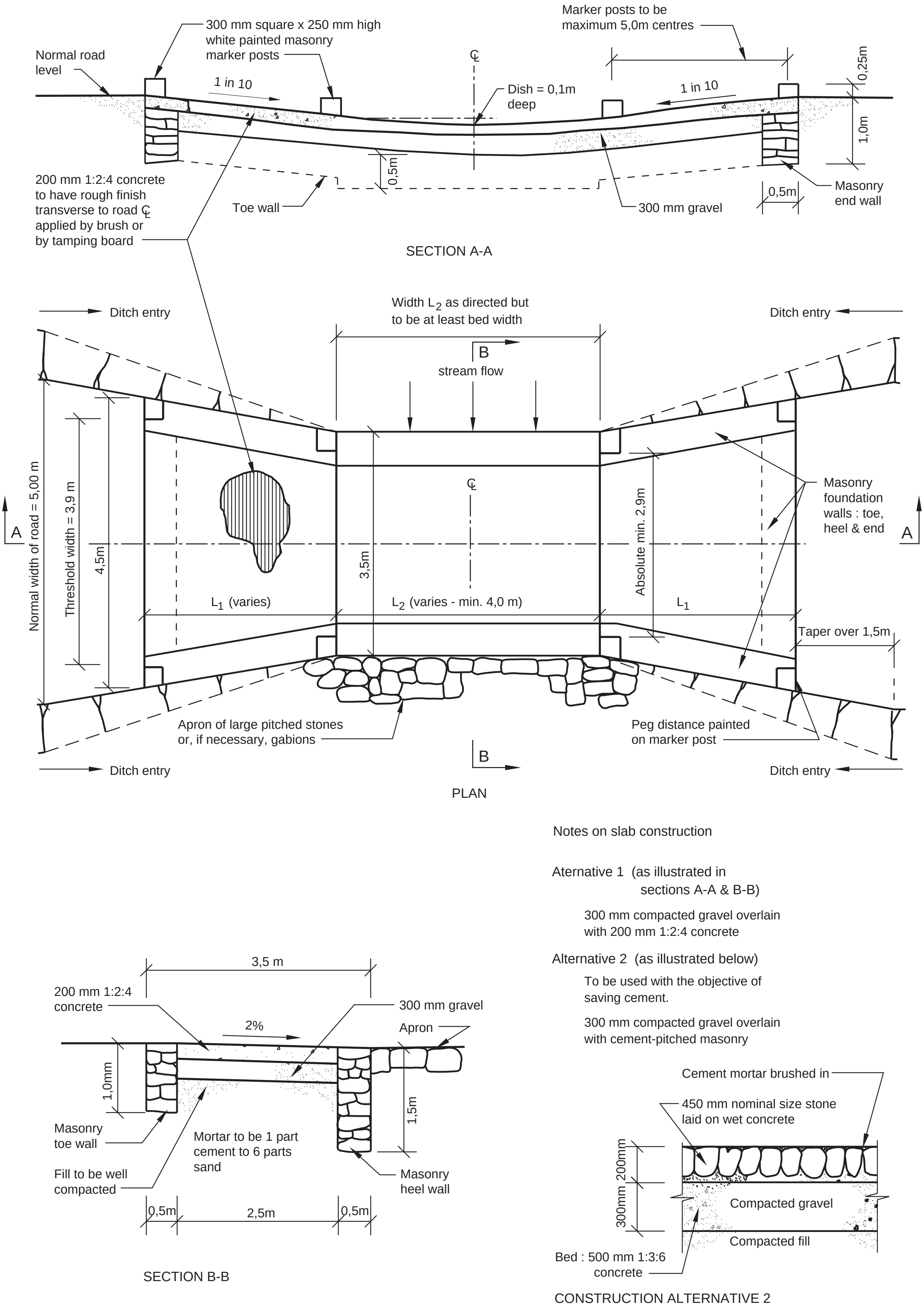


Figure 8.7: Tertiary ways: drift

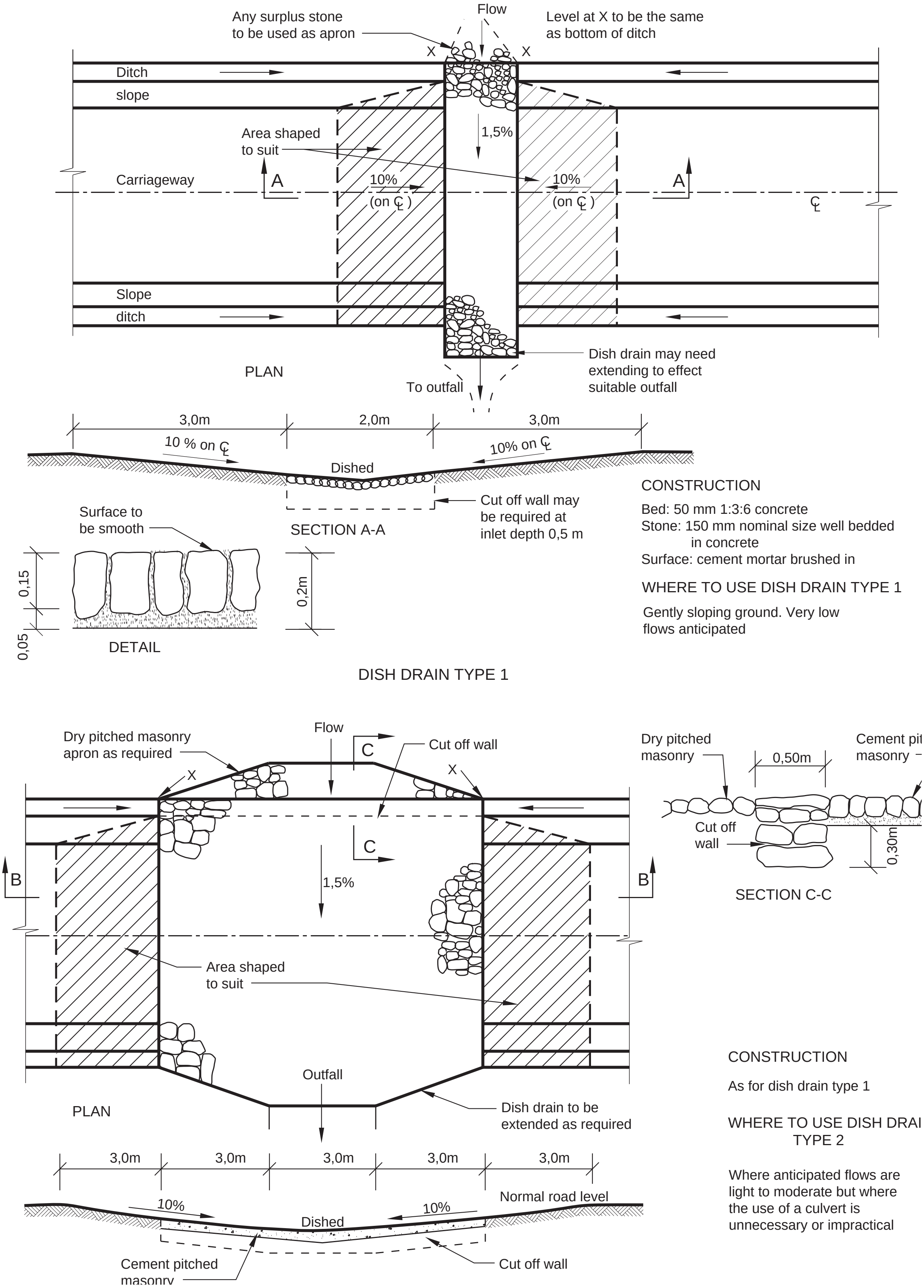


Figure 8.8: Tertiary ways: dish drains

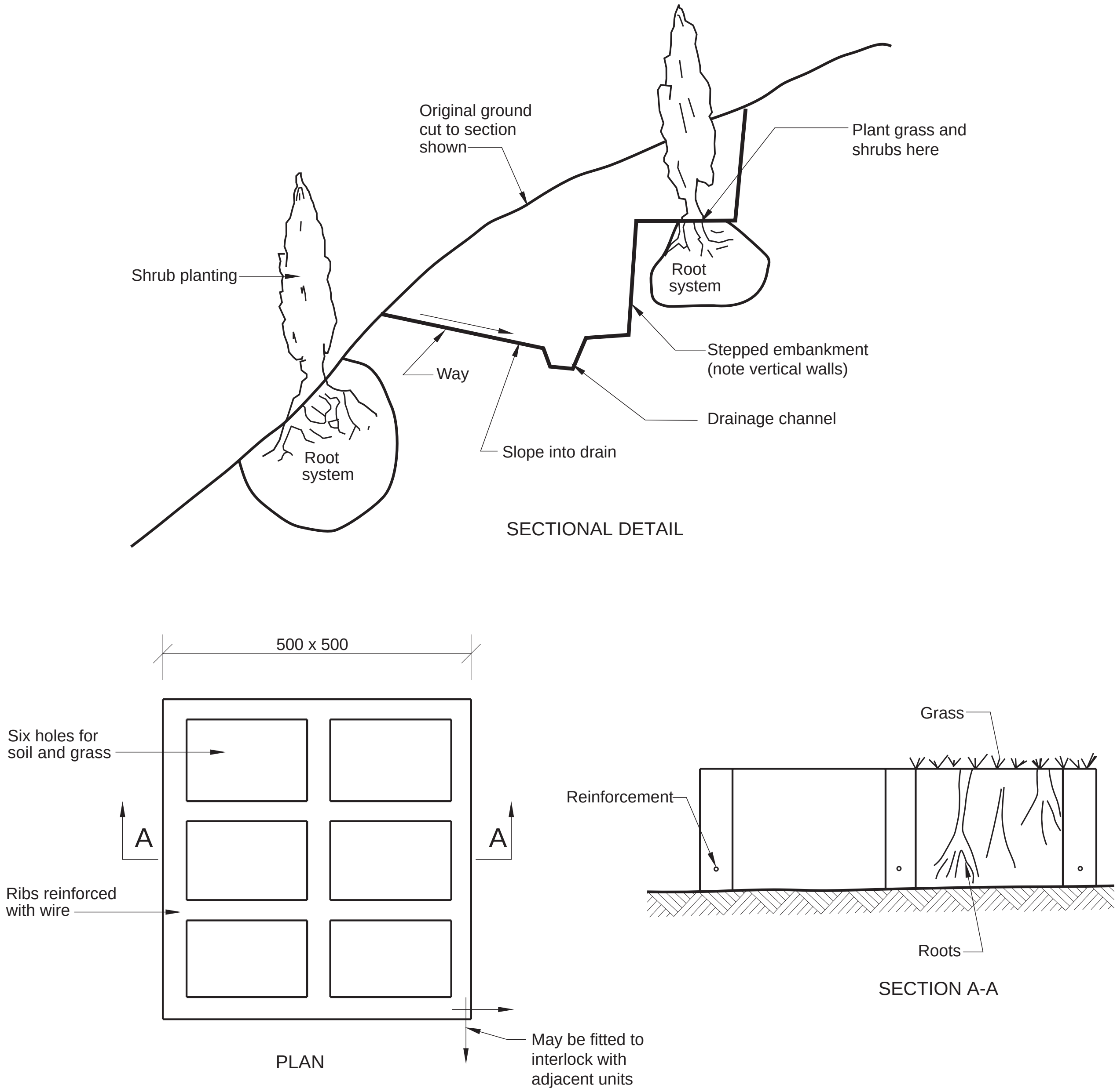


Figure 8.9: Typical grass block and vegetation

Subsurface water problems are frequently encountered where water provision in urban areas is by way of standpipes adjacent to roads. Apart from the surface runoff, significant seepage into the ground occurs and this frequently passes directly beneath the adjacent road. Careful drainage in the areas surrounding standpipes is thus essential.

As discussed earlier, subsurface drainage is a specialised field requiring a good knowledge of water flow regimes, drainage paths and filter criteria, and specialist assistance should be obtained.

Compaction

The design procedures assume that the specified material properties are satisfied in the field. A number of the traditional material properties (e.g. grading, plasticity) are independent of the construction process, but the strength is strongly dependent on the compaction achieved in the field. The design strength is based on the laboratory-determined strength of the material at a specified density. In order to ensure that this strength is obtained in the field, that particular density must be achieved during the field compaction. Table 8.14 gives the minimum compaction standards required for the various layers of the pavement structure. Note that, below base level, the standards are independent of the type of material used. As most

Table 8.14: Compaction requirements for the construction of pavement layers (and reinstatement of pavement layers)

PAVEMENT LAYER		COMPACTED DENSITY
Surfacing	Asphalt	95% 75-blow Marshall
Base (upper and lower)	Crushed stone	G1 86% to 88% apparent density G2 100% to 102% mod AASHTO
	Crushed stone G3 and gravel G4	98% mod AASHTO
	Asphalt	95% 75-blow Marshall 92% theoretical max
	Cemented	97% mod AASHTO
Subbase (upper and lower)		95% mod AASHTO
Selected subgrade		93% mod AASHTO
Subgrade (within 200 mm of selected subgrade) (within material depth)		 90% mod AASHTO 85% mod AASHTO
Fill (cohesionless sand)		90% mod AASHTO (100% mod AASHTO)

materials below base level will have a potential density somewhat in excess of the specified density which is relatively easily achieved if compaction is carried out at the correct moisture content, an attempt should be made to get as close to 100% Mod AASHTO density as possible. This has significant benefits in terms of an increased shear strength, a reduced potential to rut, and lower moisture susceptibility. Standard practice should be to roll the layer to refusal density based on proof rolling of a short section prior to its full compaction. Hand-held rollers may be inadequate to achieve the required density.

Subgrade below material depth

Special subgrade problems requiring specialist treatment may be encountered. The design procedure assumes that these have been taken into account separately. The main problems that have to be considered are the following:

- the extreme changes in volume that occur in some soils as a result of moisture changes (e.g. in expansive soils and soils with collapsible structures);
- other water-sensitive soils (dispersive or erodible soils);
- flaws in structural support (e.g. sinkholes, mining subsidence and slope instability;
- the non-uniform support that results from wide variations in soil types or states;
- the presence of soluble salts which, under favourable conditions, may migrate upwards and cause cracking, blistering or loss of bond of the surfacing, disintegration of cemented bases and loss of density of untreated bases; and
- the excessive deflection and rebound of highly resilient soils during and after the passage of a load (e.g. in ash, micaceous and diatomaceous soils).

The techniques available for terrain evaluation and soil mapping are given in TRH2 *Geotechnical and soil engineering mapping for roads and the storage of materials data* (NITRR 1978). Specialist advice should be obtained where necessary for specific problem areas. The design of embankments should be done in accordance with TRH10 *Site investigation and the design of road embankments* (NITRR 1984d).

Street levels

The fact that the provision of vehicular access adjoining streets, dwellings and commercial establishments is the primary function of an urban street means that street levels become a rather more important factor in urban areas than they are in rural or inter-urban street design. Urban street levels place some restrictions on rehabilitation and create special moisture/drainage conditions.

In some cases, rehabilitation in the form of an overlay may cause a problem, particularly with respect to the level of kerbs and channels, camber and overhead clearances. In these cases strong consideration should be given to bottom-heavy designs (i.e. designs with a cemented subbase and possibly a cemented base), which would mainly require the same maintenance as thin surfacings and little structural maintenance during the analysis period.

Urban streets are frequently used as drainage channels for surface-water runoff. This is in sharp contrast with urban, inter-urban and rural roads which are usually raised to shed the water to side table drains some distance from the road shoulder.

Service trenches

Trenches excavated in the pavement to provide essential services (electricity, water, telephone, etc) are frequently a source of weakness. This is a result of either inadequate compaction during reinstatement, or saturation of the backfill material.

Compaction must achieve at least the minimum densities specified in the catalogue of designs and material standards (Table 8.14). These densities are readily achieved when granular materials are used, but it becomes much more difficult when natural materials are used, particularly in the case of excavated clays. When dealing with clay subgrades it is recommended that, if it is economically feasible, a moderate-quality granular material be used as a trench backfill in preference to the excavated clay. In streets of Category UB and higher it is preferable to stabilise all the backfill material and in lower categories the provision of a stabilised "cap" over the backfill may be considered to eliminate settlement as far as possible. Care must be taken not to over-stabilise (i.e. produce a concrete) as this results in significant problems with adhesion of the surfacing and differential deflections causing failure around the particles.

Service trenches can also be the focal points of drainage problems. Settlement in the trench, giving rise to standing water and possibly to cracking of the surface, will permit the ingress of moisture into the pavement. Fractured water, sewerage or stormwater pipes lead to saturation in the subgrade and possibly in the pavement layers as well.

Alternatively, a trench backfilled with granular material may even act as a subsurface drain, but then provision for discharge must be made. It is, however, generally recommended that the permeability of the backfill material should be as close as possible to that of the existing layers in order to retain a uniform moisture flow regime within the pavement structure.

Pavement cross-section

Generally, it is preferable to keep the design of the whole carriageway the same, with no change in layer thickness across the street. However, where there are significant differences in the traffic carried by individual lanes (e.g. in climbing lanes), the pavement structure may be varied over the cross-section of the carriageway, provided that this is economical and practical. Under these circumstances, the actual traffic predicted for each lane should be used in determining the design traffic.

The cross-section can be varied with steps in the layer thickness, or wedge-shaped layers. Under no circumstances should the steps be located in such a way that the water can be trapped in them. Typical elements of the pavement cross-section for a paved urban street are shown in Figure 8.10.

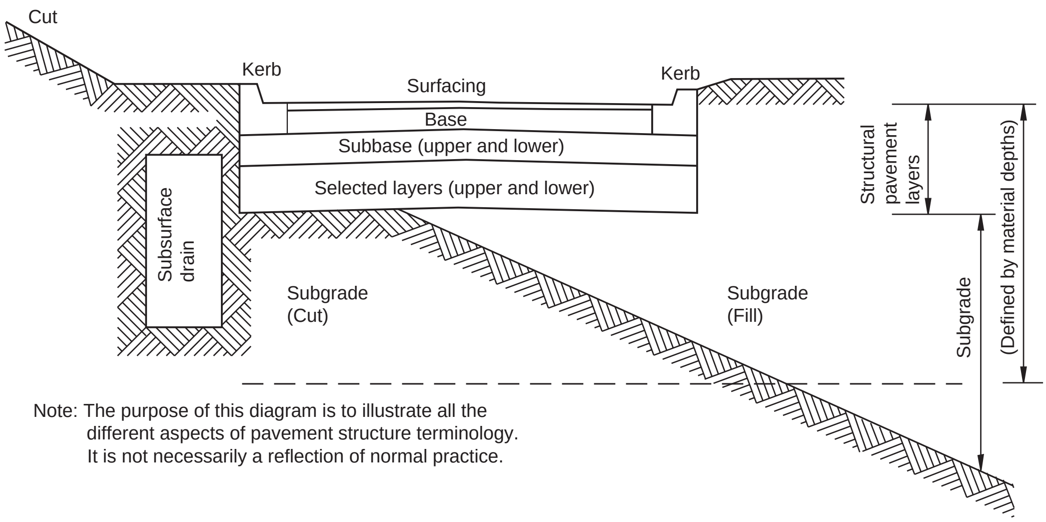


Figure 8.10: Illustrative pavement cross-section

Considerations for concrete pavements

Details on the design of concrete pavements are beyond the scope of this document. However, some basic practical recommendations are offered below (SA Department of Transport 1977):

- The subgrade should be prepared to provide a uniform support.
- The subbase should be stabilised to a high quality to provide a non-pumping, erosion-resistant, homogeneous pavement support.
- When jointed concrete pavements are used, attention should be given to joint details such as spacing, type and sealing.

Kerbs and channels

Kerbs and channels are important to prevent edge erosion and to confine stormwater to the street surface.

Consideration should be given to the type and method of construction of kerbs when deciding on a layer thicknes for the base.

It is common practice to construct kerbs upon the (upper) subbase layer to provide edge restraint for a granular base. This restraint will help to provide the specified density and strength. Care must be taken to ensure that this type of structure does not "box" moisture into the base course material.

In the case of kerbing with a fixed size (i.e. precast kerbing or kerbing with fixed shutters cast in situ) it may be advantageous to design the base thickness to conform with the kerb size (e.g. if the design calls for a 30 mm AG with a 125 mm G4 underlay, and the gutter face is 160mm, rather use a 130 mm G4).

Edging

Instead of kerbs, edging could be used for low-traffic streets when the shoulder or sidewalk material is of adequate stability. This material should be shaped to the correct level and the edge may be sealed with a prime coat, a sand seal, a slurry seal or a premix. A degree of saving may be possible by utilising trimmed grass verges where longitudinal gradients are low and stormwater flows are not likely to be high.

Accessibility

Access to dwelling units should be provided for in such a way that adequate sight distances and a smooth entry are provided, but the access ways should at the same time keep stormwater on the street from running into adjacent properties.

At pedestrian crossings special sloped openings in the kerbs should be provided to accommodate the handicapped and hand-pushed carts.

COST ANALYSIS

General

Alternative pavement designs should be compared on the basis of cost. The cost analysis should be regarded as an aid to decision-making. However, a cost analysis may not take all the necessary factors into account and it should therefore not override all other considerations. The main economic factors that determine the cost of a facility are the analysis period, the structural design period, the construction cost, the maintenance cost, the salvage value at the end of the analysis period and the real discount rate.

A complete cost analysis should be done for Category UA and UB streets. For Category UC and UD streets, a comparison of the construction and maintenance costs will normally suffice.

The method of cost analysis put forward in this document should be used only to compare pavement structures in the same street category. This is because streets in different street categories are constructed to different standards and are expected to perform differently, with different terminal levels of service. The effect these differences have on street user costs is not taken into account directly.

The choice of analysis period and structural design period will influence the cost of a street. The final decision will not necessarily be based purely on economics, but will depend on the design strategy.

The construction cost should be estimated from current contract rates for similar projects. Maintenance costs should include the cost of maintaining adequate surfacing integrity (e.g. through resealing) and the cost of structural maintenance (e.g. the cost of an asphalt overlay). The salvage value of the pavement at the end of the analysis period can contribute to the next pavement. However, geometric factors such as minor improvements to the vertical and horizontal alignment and the possible relocation of drainage facilities make the estimation of the salvage value very difficult.

Present worth

The total cost of a project over its life is the construction cost plus maintenance costs, minus the salvage value. The total cost can be expressed in a number of different ways but, for the purpose of this document, the present worth of costs (PWOC) approach has been adopted.

The present worth of costs can be calculated as follows:

$$PWOC = C + M_1 (1 + r)^{-x_j} + \ldots M_j(1 + r)^{-x_j} + \ldots -S(1 + r)^{-z} \quad (8.10)$$

where

PWOC = present worth of cost

C = present cost of initial construction

M_j = cost of the j^{th} maintenance measure expressed in terms of current costs

r = real discount rate

x_j = number of years from the present to the j^{th} maintenance measure, within the analysis period

z = analysis period

S = salvage value of pavement at the end of the analysis period, expressed in terms of the present value.

Construction costs

The checklist of unit costs should be used to calculate the equivalent construction cost per square metre. Factors to be considered include the availability of natural or local commercial materials, their expected cost trends, the conservation of aggregates in certain areas, and practical aspects such as speed of construction and the need to foster the development of alternative pavement technologies. The potential for labour-based construction also needs to be considered.

The cost of excavation should be included as certain pavement types will involve more excavation than others.

Maintenance costs

There is a relation between the type of pavement and the maintenance that might be required in the future. When different pavement types are compared on the basis of cost, these future maintenance costs should be included in the analysis to ensure that a sound comparison is made. It should also be noted that relaxations of material, drainage or pavement thickness standards will normally result in increased maintenance costs.

Figures 8.2 and 8.3 show that the life of the surfacing and water ingress into the pavement play an important part in the behaviour of some pavements. For this reason, planned maintenance of the surfacing is very important to ensure that these pavements perform satisfactorily. The service life of each type of surfacing will depend on the traffic and the type of base used. Table 8.15 gives guidelines regarding the service life that can be expected from various surfacing types. These values may be used for a more detailed analysis of future maintenance costs.

Typical maintenance measures that can be used for the purpose of cost analysis are given in Table 8.16. It should be noted that, since the costs are discounted to the present worth, the precise selection of the maintenance measure is not very important. Some maintenance measures are used more commonly on specific pavement types and this is reflected in Table 8.16. There are two types of maintenance:

- measures to improve the condition of the surfacing; and
- structural maintenance measures applied at the end of the structural design period.

The structural design period (SDP) has been defined as the period for which it is predicted with a high degree of confidence that no structural maintenance will be required. Therefore, typical structural maintenance will generally only be necessary at a later stage. If structural maintenance is done soon after the end of the structural design period, the distress encountered will only be moderate. When structural maintenance is done much later, the distress will generally be more severe. Figure 8.11 indicates the degree of distress to be expected at the time of rehabilitation for different structural design periods. Table 8.16 makes provision for both moderate and severe distress.

The typical maintenance measures given in Table 8.16 should be replaced by more accurate values, if specific knowledge about typical local conditions is available.

Street-user delay costs should also be considered, although no proper guide for their determination is readily available. The factors that determine overall street user costs are:

- running costs (fuel, tyres, vehicle maintenance and depreciation), which are largely related to the street alignment, but also to the riding quality (PSI);
- accident costs, which are related to street alignment, skid resistance and riding quality; and
- delay costs, which are related to the maintenance measures applied and the traffic situation on the streets. This is a difficult factor to assess as it may include aspects such as the provision of detours.

Table 8.15: Suggested typical ranges of period of service (without rejuvenators) of various surfacing types in the different street categories and base types (if used as specified in the catalogue)

BASE TYPE	SURFACING TYPE (≤ 50 mm THICKNESS)	TYPICAL RANGE OF SURFACING LIFE (YEARS) ROAD CATEGORY		
		A (ES3-ES100)	B (ES1-ES10)	C, D (ES0,003-ES3)
Granular	Bitumen sand or slurry seal	-	-	2 - 8
	Bitumen single surface treatment	6 - 8	6 - 10	8 - 11
	Bitumen double surface treatment	6 - 10	6 - 12	8 - 13
	Cape seal	8 - 10	10 - 12	8 - 18
	Continuously-graded asphalt	8 - 11		
	Gap-graded asphalt premix	8 - 13		
Bituminous	Bitumen sand or slurry seal	-	-	2 - 8
	Bitumen single surface treatment	6 - 8	6 - 10	8 - 11
	Bitumen double surface treatment	6 - 10	6 - 12	8 - 13
	Cape seal	-	8 - 15	8 - 18
	Continuously-graded asphalt	8 - 12	8 - 12	-
	Gap-graded asphalt premix	8 - 14	10 - 15	-
	Porous (drainage) asphalt premix	8 - 12	10 - 15	-
Cemented	Bitumen sand or slurry seal	**	-	-
	Bitumen single surface treatment	**	4 - 7	5 - 8
	Bitumen double surface treatment	**	5 - 8	5 - 9
	Cape seal	**	5 - 10	5 - 11
	Continuously-graded asphalt	**	5 - 10	-
	Gap-graded asphalt premix	**	6 - 12	-

- Surface type not normally used.
** Base type not used.

Real discount rate

When a "present-worth" analysis is done, a real discount rate must be selected to express future expenditure in terms of present-day values. This discount rate should correspond to the rate generally used in the public sector. Unless the client clearly indicates that he prefers some other rate, 8% is recommended for general use. A sensitivity analysis using rates of say 6,8 and 10% could be carried out to determine the importance of the value of the discount rate.

Salvage value

The salvage value of the pavement at the end of the period under consideration is difficult to assess. If the street is to remain in the same location, the existing pavement layers may have a salvage value but, if the street is to be abandoned at the end of the period, the salvage value could be small or zero. The assessment of the salvage value can be approached in a number of ways, depending on the method employed to rehabilitate or reconstruct the pavement.

- Where the existing pavement is left in position and an overlay is constructed, the salvage value of the pavement would be the difference between the cost of constructing an overlay and the cost of constructing a new pavement to a standard equal to that of the existing pavement with the overlay. This is termed the "residual structural value".

- Where the material in the existing pavement is taken up and recycled for use in the construction of a new pavement, the salvage value of the recycled layers would be the difference between the cost of furnishing new materials and the cost of taking up and recycling the old materials. This salvage value is termed the "recycling value".

- In some cases the procedure followed could be a combination of (a) and (b) above and the salvage value would have to be calculated accordingly.

Table 8.16: Typical future maintenance for cost analysis

BASE TYPE	TYPICAL MAINTENANCE MEASURES*			
	MEASURES TO IMPROVE THE SURFACING CONDITION**		STRUCTURAL MAINTENANCE	
	SURFACE TREATMENT ON ORIGINAL SURFACING	ASPHALT PREMIX	MODERATE DISTRESS	SEVERE DISTRESS
Granular	S1 (10 - 15 yrs) S1 (18 - 27 yrs)	S1 (12 - 20 yrs) S1 (21 - 30 yrs) or AG (13 - 22 yrs) AG (24 - 33 yrs)	30 - 40 AG, AC	>100 BS, BC or Granular overlay or Recycling of base
Bituminous		S1 (13 - 17 yrs) S1 (22 - 28 yrs) or AG (13 - 17 yrs) AG (26 - 34 yrs)	30 - 40 AG, AC	>100 BS, BC or Recycling of base
Concrete	Joints repair, surface texturing (15 yrs, 30 yrs) (equivalent cost of 20 mm PCC)		Further joint and surface repairs	Concrete granular or Bituminous overlay or Recycling
Cemented	S1 (8 - 13 yrs) S1 (16 - 24 yrs) S1 (23 - 30 yrs)	S1 (8 - 13 yrs) S1 (16 - 24 yrs) S1 (23 - 30 yrs)	Further surface treatments	Thick granular overlay or Recycling of base
Paving blocks	No maintenance measures		Re-levelling of blocks	Rebuild base, bedding sand and blocks
Cast-in-situ blocks	No maintenance measures		Remove and replace blocks with cast-in-situ blocks	Rebuild underlying layer and place cast-in-situ blocks

* S1 (10 yrs) represents a single surface treatment at 10 years and 40 AG (20 yrs) represents a 40 mm thick bitumen surfacing at 20 years.

** Refer to Table 8.15 for typical lifetimes of different surfacing types.

The salvage values of individual layers of the pavement may differ considerably, from estimates as high as 75% to possibly as low as 10%. The residual salvage value of gravel and asphalt layers is generally high, whereas that of concrete pavements can be high or low, depending on the condition of the pavement and the method of rehabilitation. The salvage value of the whole pavement would be the sum of the salvage values of the individual layers. In the absence of better information, a salvage value of 30% of initial construction cost is recommended.

Optimisation of life-cycle costs

In Table 8.2 a description of the street and its drainage is given, with their LOS values. In the matrix the combined LOS value is given. The final LOS value is determined mostly by the LOS of the drainage.

The main purpose of the determination of a representative LOS for a street is to illustrate the associated life-cycle costs. This identification can enable authorities and decision makers to select a design which will be affordable and upgradable. The costs associated with a typical street are made up of design and construction costs, maintenance costs and street-user costs. Construction costs are high for high LOS values and low for low LOS values. Maintenance costs, on the contrary, are low for high LOS values and high for low LOS values.

This concept is illustrated in Figure 8.12 with typical, present worth-of-cost versus LOS values. The combined cost curve has a typical minimum value between the highest and lowest LOS values. Street-user costs are low for high LOS streets and high for low LOS streets.

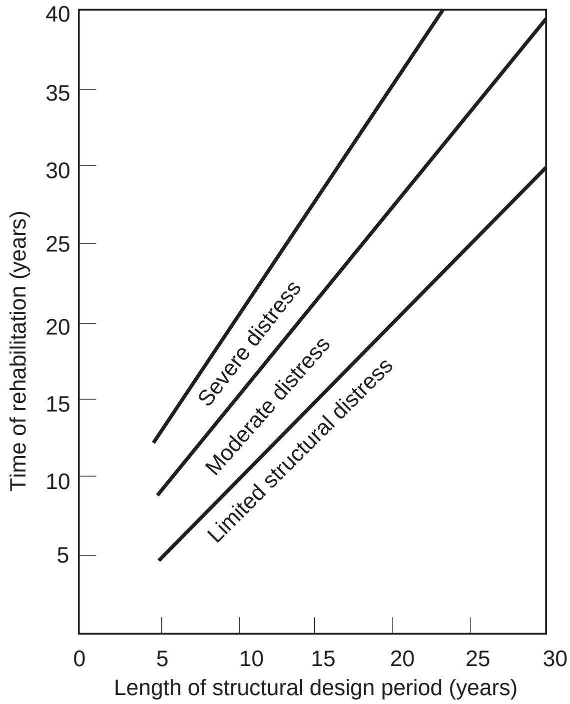


Figure 8.11: Degree of structural distress to be expected at the time of rehabilitation for different structural design periods

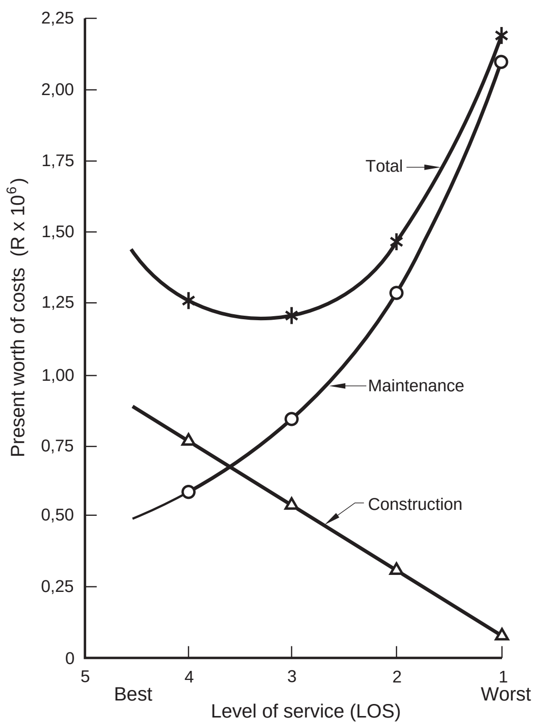


Figure 8.12: Typical cost versus level of service curve values

DISCUSSION ON THE DESIGN PROCEDURES FOR DIFFERENT STREET TYPES

At this stage the designer should have gathered enough information on the street(s) to be designed, to be able to decide which design procedure to follow - as illustrated in Figure 8.1. If an existing network is to be upgraded, the information contained in the street profiles may be used to determine paving priorities at this stage. With a background knowledge of the basic concepts from the previous section, it is now possible to go into the detailed structural design of the street pavements.

PAVED ARTERIAL AND ACCESS STREETS

The design process

The portion of the flow diagram in Figure 8.1 that refers to the design of paved arterial and access routes is enlarged upon in Figure 8.13, and divided into 8 sections. Each section will be treated separately but all sections have to be considered as a whole before a design can be produced.

The first five sections represent the basic inputs to pavement design, namely street category, design strategy, design traffic, material availability and environment. The sixth section explains how, with these as inputs, the designer can then use an appropriate design method to obtain possible pavement structures. Information on certain practical considerations in the design of streets follows in the seventh section. In the final section the analysis of alternative designs on a life-cycle cost basis, in the light of construction costs and maintenance costs, is considered.

A simplified flow diagram for the structural design of residential streets (Category UC and UD only) is suggested in Figure 8.14.

Street category

The street category will have been identified during the process of compiling the street profile, and will most likely be UA, UB or UC. The section on characteristics of streets may be consulted for a discussion on street categories in general.

Design strategy

Select an appropriate analysis and structural design period for the street under consideration. The section on street standards provides guidelines on typical analysis and design periods and lists other factors that need to be considered.

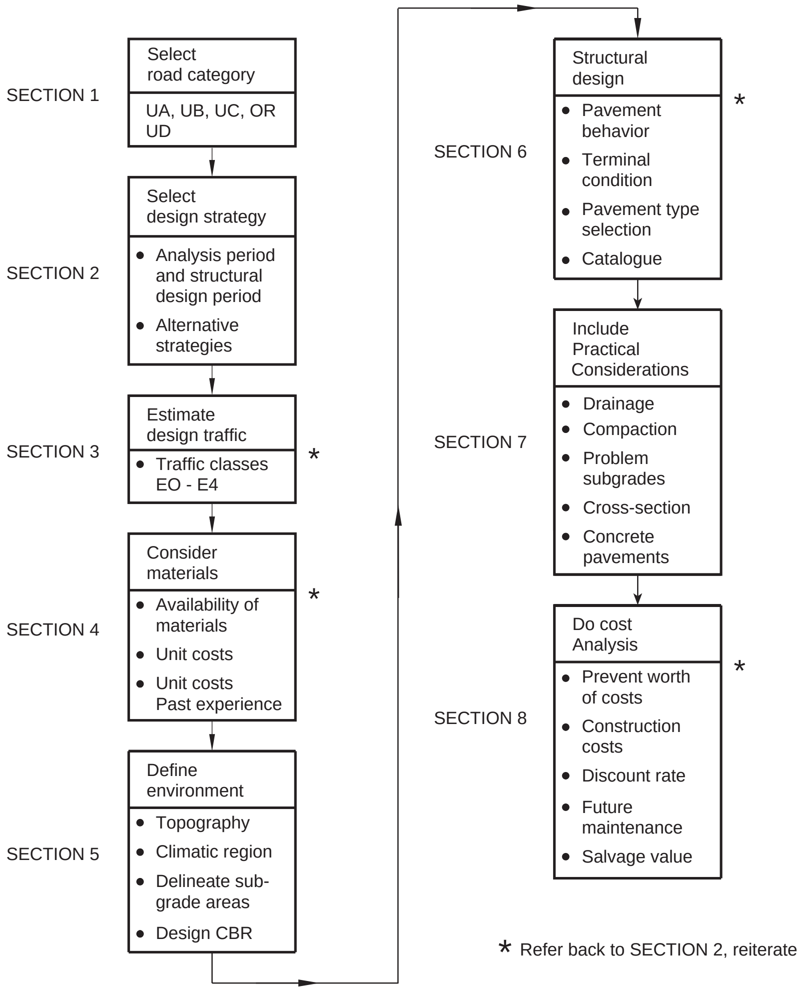


Figure 8.13: Structural design flow diagram (mainly for category UA and UB streets)

Design bearing capacity

Calculate the cumulative equivalent traffic for the particular street, according to the procedure outlined in the section on design strategy. Based on the cumulative equivalent traffic, an appropriate pavement class or design bearing-capacity interval may then be selected from Table 8.5.

Materials

Most of the materials for the selected, subbase and base layers of the pavement structures applicable to arterial and access streets will usually have to be imported. Possible material sources and the availability and cost of different types of material should be established. The availability of material combined with the expected behaviour of the major types of material and pavement, as discussed in the section on materials, will determine the final selection of the appropriate materials and pavements for particular needs.

Environment

The two most important environmental factors to consider are the climatic region and the design of the subgrade on which the street will be constructed. These are discussed in the section on environment.

Structural design

The actual structural design has two aspects - the selection of appropriate pavement types and an appropriate design method for the particular street.

<u>Pavement selection</u>

The behaviour of different pavement types has been dealt with. Certain types may not be suitable for some street categories, traffic classes or climatic regions. A number of alternative types should, however, be selected. The most cost-effective design will then be identified in the economic analysis.

Pavement structures with thin, rigid or stiff layers at the top (shallow structures) are generally more sensitive to overloading than deep structures. If many overloaded vehicles can be expected, shallow structures should be avoided.

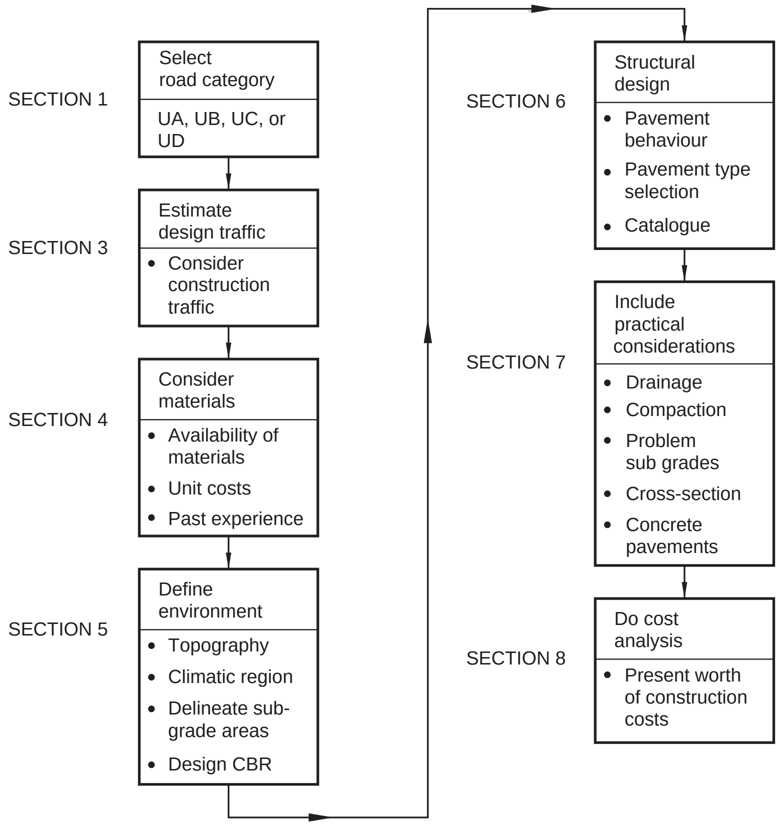


Figure 8.14: Simplified design flow diagram for residential streets (category UC and UD)

Figure 8.3 indicates that the more rigid structures deteriorate rapidly once distress sets in, whereas the more flexible pavements generally deteriorate more slowly. Signs of distress are often more visible on rigid pavements.

Pavement structures consisting of water-susceptible materials may be undesirable for wet climatic regions, unless special provision is made for drainage.

Table 8.17 shows recommended pavement types (base and subbase) for different street categories and traffic classes. Reasons why certain pavement types are not recommended are also stated briefly.

Possible condition at end of structural design period:

There is no design method available to predict the exact condition of a length of street 10 to 20 years in the future. However, certain modes of distress can be expected in certain pavement types and account must be taken of such distress. Table 8.18 shows acceptable terminal conditions of rut depth and cracking for the various street categories and pavement types. Figure 8.15 demonstrates that the rut depth values in Table 8.18 actually represent ranges of failure conditions.

Although the net depth conditions may be classified as terminal, there may be instances where the rutting has occurred primarily in the subgrade and the structural layers are still integral. In these cases the rutting may be rectified - using, for example, a thick slurry - and the street may continue to provide an acceptable level of service.

<u>Design method selection</u>

The designer may use a number of design procedures, such as the mechanistic design method, the AASHTO structural number method, the CBR cover curves or the catalogue of designs given in Appendix A. A brief overview of a selected number of design methods is given in the section on structural design methods above. Whatever the strategy used, traffic, available materials and environment must be taken into account. Some estimation of future maintenance measures is necessary before a comparison can be made on the basis of present worth of costs. Special construction considerations that might influence either the pavement structure or the pavement costs are discussed below in the section dealing with practical considerations.

This document includes the application of the catalogue design method, which is given in detail in Appendix A. However, the best results will probably be obtained if the catalogue is used together with some other design method. The

Table 8.17: Suggested pavement types for different road categories and traffic classes

PAVEMENT TYPE		STREET CATEGORY AND PAVEMENT CLASS (DESIGN BEARING CAPACITY)*										ABBREVIATED REASON WHY THE LISTED PAVEMENT TYPES ARE NOT RECOMMENDED FOR THE GIVEN STREET CATEGORY AND TRAFFIC LOADING
BASE	SUBBASE	UA			UB		UC			UD		
		ES 30	ES 10	ES 3	ES 3	ES 1	ES 1	ES 0,3	ES 0,1	ES 0,1	ES 0,03	
Granular	Granular	✗	✗	✓	✓	✓	✓	✓	✓	✓	✓	Uncertain behaviour
	Cemented	✓	✓	✓	✓	✓	✓	✓	✓	✓	✗	
Asphalt hot-mix	Granular	✗	✗	✗	✗	✗	✗	✗	✗	✗	✗	
	Cemented	✓	✓	✓	✓	✓	✗	✗	✗	✗	✗	
Concrete	Granular	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	Extra thickness required to prevent fatigue cracking
	Cemented	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	Too expensive, too difficult to trench
Cemented	Granular	✗	✗	✗	✗	✗	✗	✓	✓	✓	✓	Fatigue cracking, pumping and rocking of blocks
	Cemented	✗	✗	✗	✓	✓	✓	✓	✗	✗	✗	Shrinkage cracks unacceptable
Paving blocks	Granular	✗	✗	✗	✓	✓	✓	✗	✗	✗	✗	Not recommended at high speeds
	Cemented	✗	✗	✗	✓	✓	✓	✓	✓	✓	✓	
Bituminous cold-mix	Granular	✗	✗	✗	✓	✓	✓	✓	✓	✓	✓	
	Cemented	✗	✓	✓	✓	✓	✓	✗	✗	✗	✗	
Macadams	Granular	✗	✗	✗	✗	✓	✓	✓	✓	✓	✓	
	Cemented	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	
Cast-in-situ blocks		✗	✗	✗	✓	✓	✓	✓	✓	✓	✓	Not recommended at high speeds

See Table 8.6 for definition of pavement classes.

Table 8.18: Possible condition at end of structural design period for various street categories and pavement types

POSSIBLE CONDITION AT END OF STRUCTURAL DESIGN PERIOD	ROAD CATEGORY			
	UA	UB	UC	UD
Rut depth Length of road exceeding stated rut depth (refer to Figure 8.15)	20 mm 10 %	20 mm 15 %	20 mm 25 %	20 mm 40 %
Type of cracking				
Granular base	Crocodile cracking, surface loss, pumping of fines			
Bituminous base	Crocodile cracking, pumping of fines			
Concrete pavement	Slab cracking, spalling at joints, pumping of fines			
Cemented base	Block cracking, rocking blocks, pumping of fines			
Proportion of road on which stated types of cracking occur	10%	15%	25%	40%

catalogue method can serve as a useful starting point, even if other design methods are used.

The application of the catalogue design method

General:
It should be noted that these designs are considered adequate to carry the total design equivalent traffic over the structural design period. Construction constraints on practical layer thicknesses and increments in thicknesses are met. It is assumed that the requirements of the material standards are met. The catalogue may not be applicable when special conditions arise; other methods should then be used, but the catalogue can still act as a guide. The catalogue does not necessarily exclude other possible pavement structures.

Selected layers:
The catalogue assumes that all subgrades are brought to equal support standards. The design CBR of the subgrade is limited to four groups (Table 8.12). Normally, the in situ subgrade soil will be prepared or ripped and recompacted to a depth of 150 mm. On top of this prepared layer, one or two selected layers may be added. The required selected subgrade layers will vary, according to the design CBR of the subgrade. Table 8.19 shows the preparation of the subgrade and required selected layers for the different subgrade design CBRs.

Interpolation between traffic classes:
The pavement structures in the catalogue are considered adequate to carry the total design traffic, according to the upper value of the traffic classes defined in Table 8.5. The total design traffic may be predicted with more accuracy than is implied by the traffic classes. In such a case the designer may use a simple linear interpolation technique. In many designs the only difference between the structures for the various classes of traffic is a change in the layer thickness. In these cases the designer may use linear interpolation. However, there is often a change in material quality, as well as in thickness. Simple interpolation is then inadequate and the designer will have to use other design methods.

Surfacings:
Urban and residential streets carry both traffic and stormwater runoff. The traffic often consists of either large volumes of lightly loaded vehicles (e.g. on CBD streets) or virtually no vehicular traffic (e.g. on residential streets and culs-de-sac). In both cases, however, a high-quality surfacing is required. Such surfacing is also necessary because the street acts as a water channel.

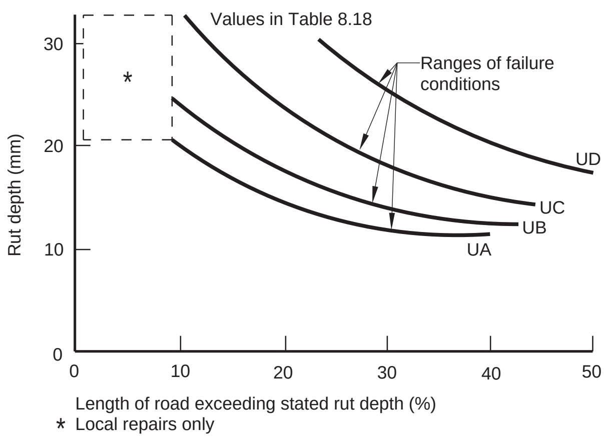


Figure 8.15: Ranges of terminal rut depth conditions for different street categories

Table 8.19: Preparation of subgrade and required selected layers for the different subgrade design CBRs*

DESIGN CBR OF SUBGRADE	<3	3 - 7	7 - 15	15 - 25	>25
Add selected layers: Upper Lower	Not applicable	150 mm G7 150 mm G9	150 mm G7 -	- -	- -
Treatment of in-situ subgrade	Special treatment required	Rip and recompact to 150 mm G10	Rip and recompact to 150 mm G9	Rip and recompact to 150 mm G7	Use subbase or base layer**

* Not applicable to category UD roads: for these use only one selected layer (G7) if required.
** Compacted to the appropriate density (see Table 8.14).

The factors influencing the choice of surfacing are:

- local experience;
- availability;
- street category;
- design traffic class;
- environment (e.g. moisture, temperature and ultraviolet radiation);
- pavement type and deflection;
- maintenance capability;
- turning movements, intersections, braking movements; and
- gradients.

The catalogue specifies the surfacing type, but allows a choice of surfacings for the lower categories of street. The controlling authority should select a surfacing from the catalogue that will give satisfactory performance (SABITA 1993).

If a waterbound macadam is used in the base in the place of a GI to G4 material, the thin surfacings will be inadequate to provide acceptable riding quality on the coarse surface typically obtained. In such cases, up to 50 mm of asphalt premix may be required.

Practical considerations

A host of practical issues that need to be considered are covered in the section on practical considerations above.

Economic analysis

The purpose of the economic analysis is to identify the most economically viable design. The economic analysis is strongly linked to the design strategy and the life-cycle cost of the alternative designs. In the case of category UC and UD streets, where a design strategy is not necessarily formulated, only the construction cost needs to be considered for designs without a design strategy.

PAVED BASIC ACCESS STREETS

The functional classification of basic access streets (see street categories under “compiling a street profile” above) indicates that traffic volumes are so low that the traditional design guidelines are not applicable in most cases. (CUTA 1988b). Non-traffic related factors such as layout planning, stormwater management and drainage, climate, environment, topography and in- situ materials have a major influence on the design of basic access streets. Adherence to the guidelines for layout planning and stormwater management (CUTA 1988a) is a prerequisite for the sound design of basic access streets.

As mentioned in the earlier section on the drainage of basic access streets, there may be a number of reasons for paving a basic access street. The decision process is illustrated in Figures 8.1 and 8.16.

If erosion problems are identified, erosion protection must be supplied. Dust palliatives may be considered as one option in this regard.

Structural design of paved basic access streets

Although basic access streets normally carry light traffic, the final desired pavement structure should be designed and constructed only after the infrastructure development is completed to avoid damage by construction traffic. The street can be constructed up to subbase level and used in an unpaved state during infrastructure development, before correction of the subbase and application of the base and surfacing.

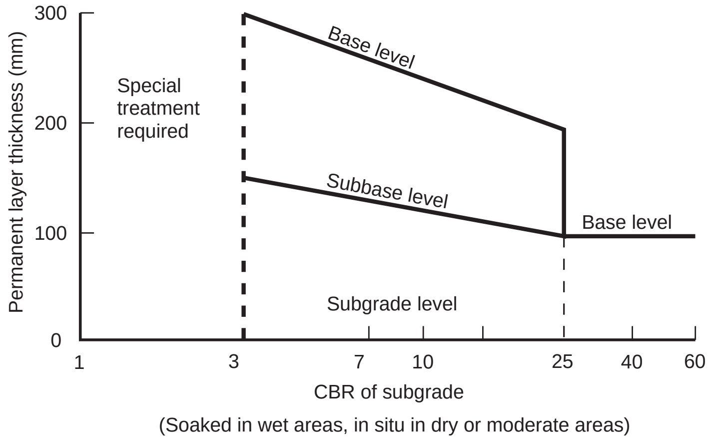


Figure 8.16: Pavement design curves for basic access streets

Selection of pavement type

Because of the very low bearing capacities for which basic access streets are designed, the pavement structures of these streets will be very light. Coal and refuse-removal lorries, however few, are bound to be overloaded with destructive effects on surfacings and the pavement structure. This should be recognised in the design phase and pavement types sensitive to overloading should not be selected.

Selection of design method

The dynamic cone penetrometer (DCP) design method, the catalogue in Appendix A, or the design method described below could be used for the structural design of paved basic access streets.

Pavement structures are bound to be mostly granular on basic access streets and therefore the use of the DCP model (Kleyn and van Zyl 1987) is strongly recommended. The minimum number of blows to penetrate 800 mm of the pavement structure is 110 in a dry condition and 60 in a wet condition (Kleyn 1982). Streets with even fewer pavement structural layers than the traditional three have been found to perform satisfactorily as basic access streets (Horak 1988), but designs should be checked with the DCP model.

Figure 8.16 can also be used as a guideline for the design of the pavement structure. It is suggested that a soaked CBR be used only in wet areas while in-situ CBR values from DCP soundings can be used in moderate to dry areas (Emery 1984). The quality of in-situ material will determine the need for a subbase layer. Where an asphalt surfacing of 25mm is used, the base thickness may be reduced by the surfacing thickness, provided that the deflections do not increase to the level that rapid fatigue failure of the asphalt occurs.

The implications and problems with relaxing material specifications for paved streets have been fully discussed by Netterberg and Paige-Green (1988). Each layer in the street should be treated as a separate entity with respect to the materials used and the construction procedures.

Selected layers:
If the in-situ material is not of at least G9 quality (NITRR 1984c) material of this quality or better should be imported and placed on the compacted in-situ material. If the in-situ material is of G9 or better quality, no selected layer is necessary. The subgrade or selected layers should be compacted to at least 93% Mod AASHTO density, but preferably to refusal density.

A summary of the recommended quality of the platform for the subbase is therefore as follows:

- min CBR at OMC and 90% Mod AASHTO compaction: 7;
- min. field compaction: 93% Mod AASHTO.

Subbase:
Once the foundation of the subbase has a CBR of at least 7 (i.e. at least G9), material of G6 or better quality should be used for the subbase. The moisture content at which the CBR values are determined should be near the expected field moisture content (OMC is often considered a reasonably conservative estimate).

The following specifications are recommended for subbase material:

- minimum CBR at OMC and field compaction: 25;

- min compaction: 95% Mod AASHTO, preferably refusal;
- max size: two-thirds of layer thickness.

Specifying the material strength makes it unnecessary to limit properties such as grading (of the matrix in particular) and plasticity, as it is these properties which determine the strength at any specific moisture content and density.

It is important to carry out the CBR test in a standard manner and the recommended method is that all oversize material (larger than 19,0mm) is discarded. This procedure is considered to result in a slightly conservative CBR as the larger material would generally produce more interlock and shear resistance.

In many cases where the in-situ material has a CBR of 25 or more, no subbase is necessary and the base can be applied directly onto the in-situ material compacted to at least 95% Mod AASHTO density.

Base:
The base is the most important structural layer of a lightly trafficked street, and must have adequate strength and durability to perform satisfactorily during the life of the street. Structurally, a material with a CBR strength of 30 to 50 at field moisture and density is adequate for lightly trafficked streets (Kleyn and Van Zyl 1987). A compaction of 98% Mod AASHTO is necessary to limit traffic-associated compaction to an acceptable amount, although it should preferably be compacted to refusal density. The durability of the material, however, must be ascertained for many aggregates - especially basic igneous rocks.

It is considered necessary to provide different requirements for each material group. The bearing capacity is the major criterion in each case, and this should be related to the prevailing environmental conditions. The bearing capacity of bases for lightly trafficked streets is best specified in terms of the CBR value at 98% Mod AASHTO compaction at the expected field moisture conditions. In poorly drained and wet areas it may be necessary to use the soaked values, while in arid areas a test at the OMC (Emery 1984) may be used for the design. In South Africa the ratio of the equilibrium moisture content (EMC) in the base to the optimum moisture content (OMC) seldom exceeds 0,6 (Emery 1984).

The risk of using an unsoaked CBR in the subgrade for the design can be quantified and expressed as the probability of the street failing before the design traffic has been carried for varying moisture contents (Emery 1987). The example discussed by Emery (1984) indicates that, even with an EMC/OMC ratio of more than 1,5, the probability of the street not carrying 0,2 M E80s is less than about 6%.

UNPAVED ARTERIAL AND ACCESS STREETS

The most common causes of poor performance of gravel streets are slipperiness and potholing when wet, and producing excessive dust and ravelling when dry. The formation of corrugations is normally the result of inadequate compaction or low cohesion combined with traffic. Frequent maintenance (e.g. grading, watering and the addition of material) is necessary and the frequency of maintenance will increase with increasing traffic.

Street category

Normally, unpaved streets could be considered for use as Category UC or UD streets, although there may be special cases where they can be regarded as Category UB streets.

Design strategy

A gravel street may be regarded as a long-term facility or as an interim step towards a paved street. This will influence the level of the surface with regard to stormwater facilities and geometric considerations.

Materials

Material will usually have to be imported for at least the wearing course of gravel arterial and access streets. Possible material sources must be identified early in the design process.

Design of imported layers

Selected layers for gravel streets

The selected layers for gravel streets should preferably be designed as for paved arterial and access streets, although this is not critical as discussed earlier. The minimum subgrade CBR, however, should be 3 for less than 100 vpd and 5 for more than 100 vpd, or else 150 mm of “subbase” with a CBR of 5 and 10 respectively should be imported to support the base. It is advisable, however, that at subgrade level there is no distinction between paved and unpaved streets, to simplify possible later changes from unpaved to paved streets.

Design of gravel wearing course

The standards for gravel wearing courses are laid down in Appendix B. The quality and thickness of the wearing course may also depend on the design approach, as follows:

- The gravel street may be regarded as a long-term facility, in which case the most suitable wearing course will be selected for the prevailing conditions (climate, material availability and traffic). On more heavily trafficked streets, the gravel wearing course should be used as a future subbase. On access streets it could constitute the future base course.

- The gravel wearing course may be regarded as an interim riding surface, which will be overlaid or removed when the street is paved. If the gravel wearing course is going to be overlaid later, the material should comply with the subbase standards applicable to the future pavement. If the wearing course is to be removed later, consideration should be given to the inclusion of a proper subbase during construction, should such a subbase be necessary.

- The gravel wearing course may be regarded as the base layer of the future paved street. This will normally be possible only for some Category UC or UD streets, but then special restrictions may be placed on the plasticity index of the fines.

The thickness of the gravel wearing course will usually be a standard 150 mm. Passability during the wet season is best determined by the soaked laboratory CBR of the surfacing gravel material. Figure 8.17 gives a proposed limit related to the average daily traffic (ADT).

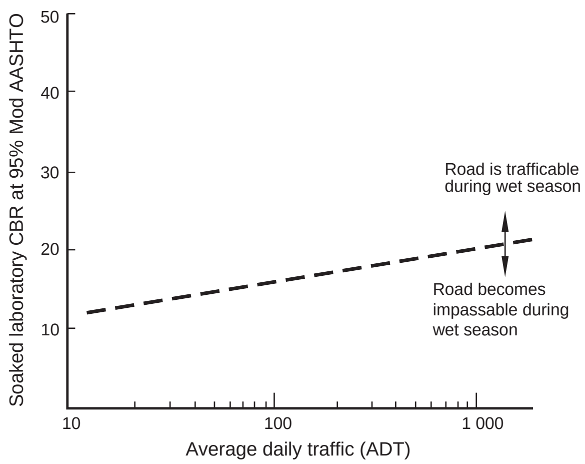


Figure 8.17: Design curves for the passability of unpaved roads

When a gravel wearing course has to be provided because the existing subgrade cannot support the traffic loads adequately under all climatic conditions, the thickness requirement for the gravel wearing course can be determined analytically. A thickness design method which gives the required material thickness for adequate subgrade protection, regulated gravel loss and deficiencies in compaction is described in TRH20.

UNPAVED BASIC ACCESS STREETS

The functional classification of basic access streets (see section on categories of street) indicates that traffic volumes are so low that the traditional design guidelines are not applicable in most cases. (CUTA 1988b). Non-traffic related factors such as layout planning, stormwater management and drainage, climate, environment, topography and in situ materials have a major influence on the design of basic access streets. Adherence to the guidelines for layout planning and stormwater management (CUTA 1988a) is a prerequisite for the sound design of basic access streets.

Figures 8.1 and 8.18 illustrate the decision process for the design of basic access streets. If the decision is taken not to pave a basic access street, attention should be paid to erosion protection, the quality of the in-situ material and the quality of the wearing course (if required).

Erosion-prone in-situ materials should be identified (Rooseboom and Mulke 1982). The length of erosion-free in situ material can be determined if the gradient and basic material information is available. If erosion problems are identified, erosion protection must be provided. Surface stabilisers may be considered in this regard.

As shown in Figure 8.18, in-situ material can be used without a surfacing if it meets the appropriate material standards for basic access streets (TRH20).

The thickness of a gravel wearing course should be 100mm unless experience suggests otherwise. If in-situ material meets the specified material requirements of TRH20, a gravel wearing course need not be imported to the street (Netterberg 1985).

TERTIARY WAYS

As settlements develop, so does an infrastructure of narrow ways which carry predominantly pedestrians, cattle and bicycles. These non-trafficked (motorised traffic) tertiary ways are mostly informal and unserviced, but in older settlements in developing areas they are formalised (upgraded) by the provision of surfacings, drainage or even services like water and electricity (Clifford 1987).

Layout

In an informal settlement development no layout planning for tertiary ways is done. The existence of these ways is dictated by pedestrians' need to follow the shortest possible routes. At a later stage of

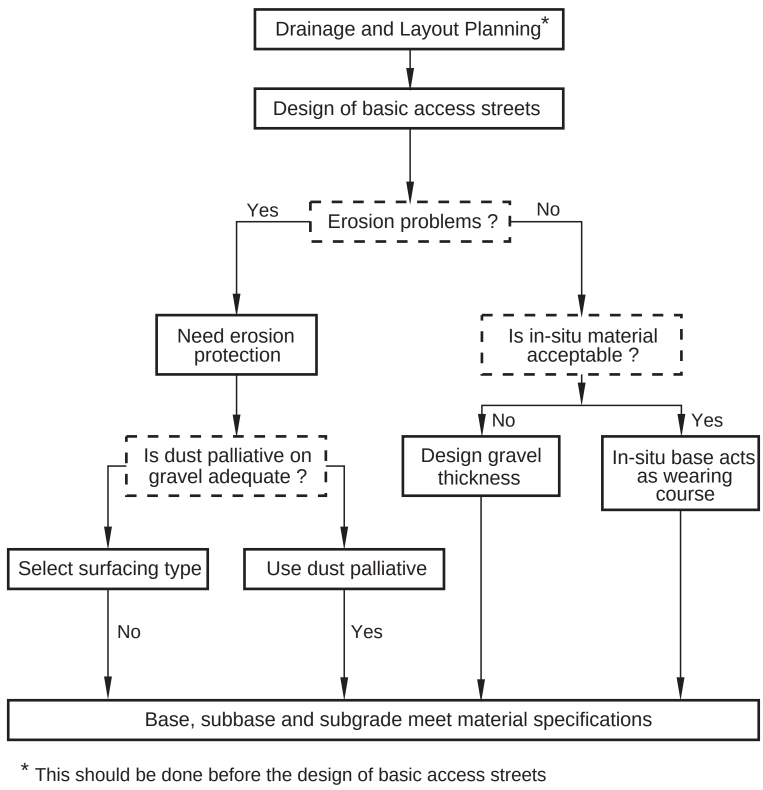


Figure 8.18: Flow diagram of design process for basic access streets

development some of these tertiary ways may be replaced by basic access streets. It is advisable to design for tertiary ways during layout planning as these form the basic links between dwellings. In a formal design they can enhance the design principles applicable to the higher order of streets.

Material

The in-situ material is mostly used for tertiary ways. However, no standards are applicable, as no formal construction is carried out during the initial informal development stages. In general, where problem materials exist, they should be covered or improved to ensure passability. The discussion on material types covered in Appendix C is applicable to the identification of possible material problems.

Design of tertiary ways (standard cross-sections)

No formal design exists during the early stages of growth in a developing community. In the later stages of development these tertiary ways can be upgraded to have an appropriate profile, with side drains. In such cases, and for those tertiary ways which are designed from the outset, various cross-sections are suggested for various ground slopes (Figure 8.19). The cross-slopes of a tertiary way are limited to 10%. In the case of very steep sloping ground, the camber is replaced by a 7% cross-slope against the ground slope. Dimensions are also given for ditches, berms and retaining walls. In Figure 8.20, typical cutting and embankment cross-section details are given, with a table on embankment details.

Dust palliatives

Unacceptable levels of dust are experienced on many of these roads, especially those in rural and urban communities. In the past, dust was only considered a nuisance factor. However, recent studies have indicated that the dust generated by vehicles on unpaved roads could have significant environmental and social impacts in terms of health and safety issues, visual pollution, and economic impacts pertaining to loss of road construction material, increased building maintenance and higher vehicle-operating costs.

Consequences of dust

The main consequences of dust include discomfort for pedestrians, vehicle occupants and residents of properties adjacent to the road. Visibility for following and approaching vehicles is greatly reduced, creating a safety risk for motorists, cyclists, pedestrians and livestock. In addition, vehicle-operating costs increase dramatically on dusty roads. Other important effects of dust include health hazards, reduced agricultural yields, pollution and loss of road construction material.

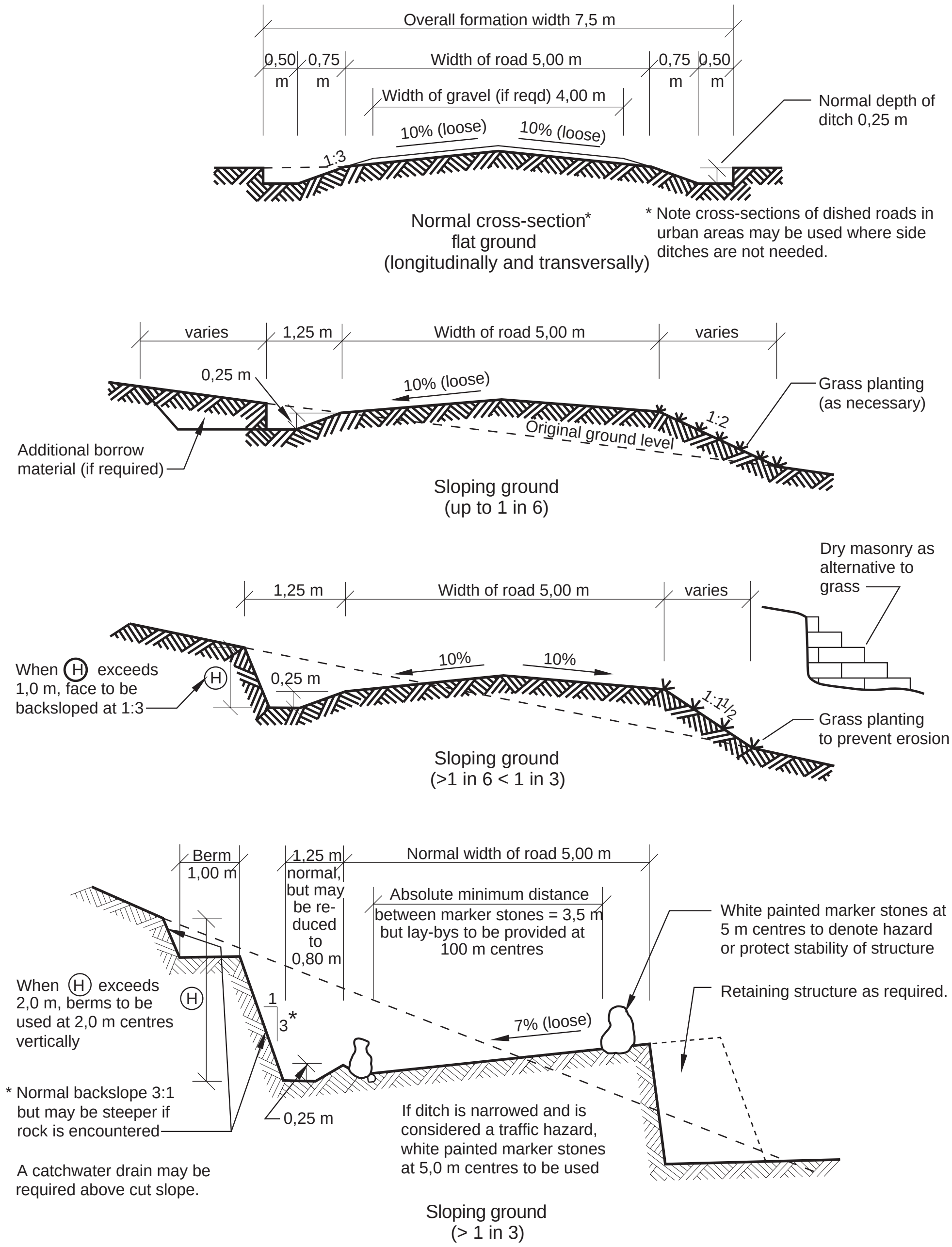


Figure 8.19: Tertiary ways: cross-sections

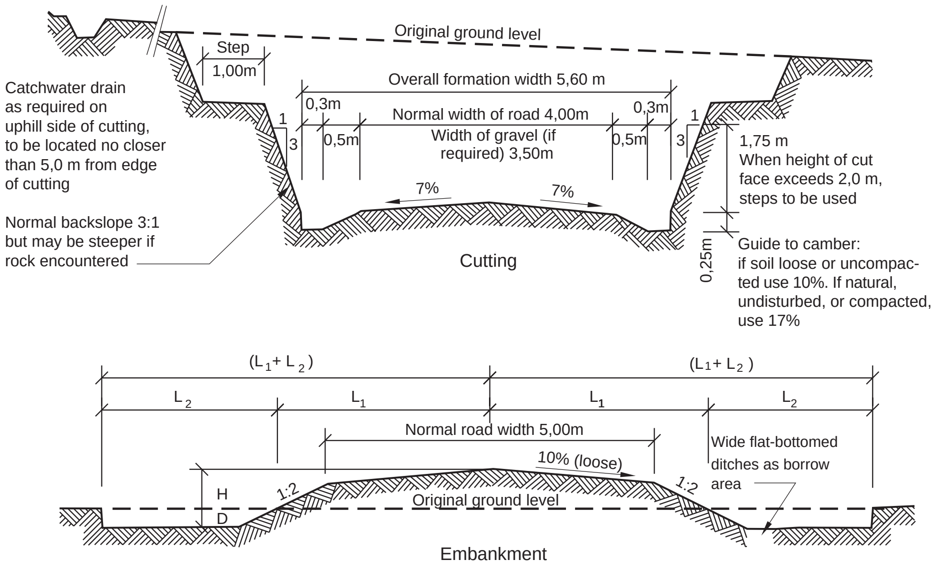
Processes affecting the generation of dust

Dust generation is predominantly related to the silt content, plasticity characteristics, hardness and relative density of the aggregate, and the threshold shear velocity of the wind generating the dust. The moisture content of the material and the period since the road was last bladed will also influence the level of dust. The parameters used in dust prediction include plasticity index, linear shrinkage, percentage passing 0,075 mm, the aggregate impact value and the relative density.

Dust control

In terms of the total unpaved road network in South Africa, minimal dust control is currently being carried out. The last few years have seen a proliferation of products which the manufacturers claim will reduce both dust and maintenance on unpaved roads. However, minimal specification of their properties or records of their performance are available and very few properly controlled comparative tests on the effectiveness of the products from different producers and suppliers have been carried out in full-scale field trials.

Research on dust palliatives has been conducted by the CSIR over a period of six years on behalf of road authorities and product manufacturers. In order to facilitate the selection of an appropriate product for particular conditions, dust palliatives are divided into the following ten categories:



Embankment dimensions											
Embankment height H	Area of embankment A	Depth of ditch D	Offset from ℄ L_1	Offset to edge L_2	Total offset (L_1+L_2)	Embankment height H	Area of embankment A	Depth of ditch D	Offset from ℄ L_1	Offset to edge L_2	Total offset (L_1+L_2)
0,30	2,12	0,30	3,10	3,83	6,93	1,10	8,36	0,50	4,70	10,20	14,30
0,35	2,43	0,30	3,20	4,35	7,55	1,20	9,32	0,50	4,90	10,61	16,28
0,40	2,76	0,30	3,30	4,90	8,20	1,30	10,32	0,50	5,10	11,37	17,60
0,45	3,09	0,30	3,40	5,45	8,85	1,40	11,36	0,50	5,30	12,50	18,81
0,50	3,44	0,30	3,50	6,03	9,53	1,50	12,44	0,50	5,50	13,51	20,10
0,55	3,79	0,40	3,60	5,14	8,74	1,60	13,56	0,60	5,70	14,60	22,52
0,60	4,16	0,40	3,70	5,60	9,30	1,80	15,92	0,75	6,10	16,82	25,50
0,65	4,53	0,40	3,80	6,06	9,86	2,00	18,44	0,75	6,50	19,41	27,82
0,70	4,92	0,40	3,90	6,55	10,45	2,20	21,12	1,00	6,90	21,32	30,23
0,75	5,31	0,40	4,00	7,04	11,04	2,40	23,96	1,00	7,30	23,33	32,50
0,80	5,72	0,40	4,10	7,55	11,65	2,60	26,96	1,25	7,70	25,20	34,90
0,85	6,13	0,40	4,20	8,06	12,26	2,80	30,12	1,25	8,10	27,20	37,20
0,90	6,56	0,40	4,30	8,60	12,90	3,00	33,44	1,25	8,50	29,10	43,50
0,95	6,99	0,40	4,40	9,14	13,54	3,50	42,44	1,50	9,50	34,00	49,32
1,00	7,44	0,40	4,50	9,70	14,20	4,00	52,44	1,75	10,50	38,82	49,32

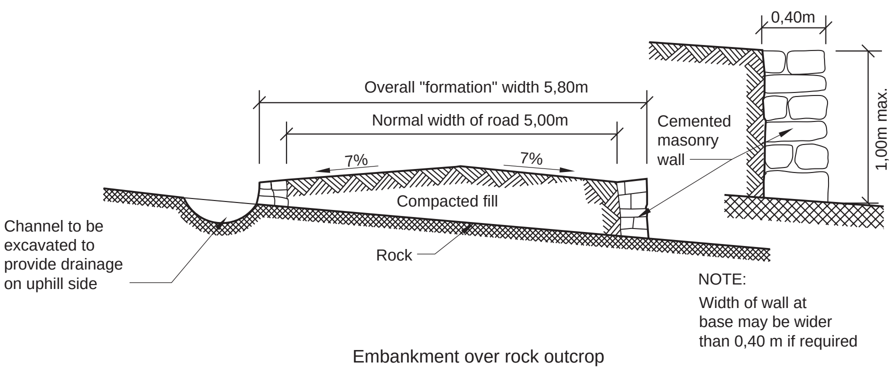


Figure 8.20: Labour-intensive tertiary ways: cross-sections, cuttings and embankments

- water and wetting agents;
- waste oils;
- emulsified petroleum resins;
- hygroscopic materials;
- ligno-sulphonates;
- modified waxes;
- acrylate polymers;
- tars and bitumens;
- sulphonated oils; and
- other products.

Environmental considerations

Growing environmental awareness in the roads industry has necessitated the development of appropriate tests for assessing the impact of spraying chemical dust palliatives onto unpaved roads. A series of interim test methods to determine the toxicity of additives to surface water, groundwater and vegetation has been developed and is currently being perfected. Initial results indicate that all the products are toxic to aquatic life in an undiluted form, and standard precautions for the handling and transportation of chemicals should be adhered to. Testing of the leachate from soils to which the product has been applied at the recommended application rates indicated that none of the leachates from any of the products tested is likely to influence groundwater or surface water. Certain products are likely to kill plants if they are sprayed directly onto the leaves. However, root uptake of the leachate by plants is unlikely to cause death.

CONSTRUCTION

Staged construction and upgrading

Two concepts that need to be considered as part of the life-cycle strategy of a street during design are staged construction and upgrading. Although it is difficult to exactly define and completely separate these concepts, certain characteristics may be more typical of one than of the other.

The aim of staged construction is to spread some of the financial load from the initial construction period to some stage later during the life cycle of the facility. However, right from the onset, the aim is to provide a particular level of service for the duration of the structural design and analysis period of the facility. There may be slight changes in, for instance, the riding quality of the facility, but these should have a marginal influence on the operating cost of the facility to the user.

On the other hand, upgrading will normally take place when the demands placed on an existing facility far exceed the level of service the facility can provide. The new facility has to provide a much higher level of service at a much reduced cost to the user. An example would be the upgrading from a gravel to a surfaced street.

Staged construction may be done by adding a final layer, or reworking an existing layer at some stage early during the structural design period of the facility. Most of the money spent during the initial construction of the facility should therefore be invested in the lower layers of the structure, providing a sound foundation to build on in the future.

During the upgrading process, maximum use should be made of the existing foundation provided by the pavement being upgraded. Dynamic cone penetrometer (DCP) and impulse deflection meter (IDM) surveys may provide the information required to incorporate the remaining strength of the existing pavement in the design of the future facility. Special equipment may also be used to maximise the bearing capacity of the in-situ material. Impact roller compaction can prove very useful in the urban environment. With this equipment, it is usually possible to compact the in-situ material to a depth of 600mm at densities well above those normally specified for the subgrade and selected layers of a pavement, without excavating and replacing any material. This results in few layers or thinner layers being required in the pavement structure.

One of the problems that may be associated with staged construction in the urban environment is the limitation placed on street levels by the other services in the street reserve, particularly the stormwater drainage system. If a system of kerbs, gutters and stormwater pipes is used, it may not be possible to add an additional structural layer to the pavement system at a later stage. In such cases, consideration should be given to initially providing a subbase-quality gravel base, and to rework this layer at some early stage in the structural design period of the pavement by doing deep in-place recycling and stabilisation with cement, bitumen emulsion or a combination of the two. A second problem that requires consideration is the cost of repeated mobilisation on a project. The mobilisation of plant and resources for a light pavement structure in an urban area (usually of short length) is often a significant portion of the total cost.

Table 8.20 provides a number of staged construction examples developed from the catalogue of pavement designs contained in Appendix A. Both the initial and final pavement structures are illustrated for the staged construction options. The staged construction of these examples is not achieved by merely taking a design for a high bearing capacity and removing the base layer. The structures for the initial construction phase are actually also taken from the catalogue and have a known bearing capacity. This should assist in determining the time allowed to elapse before completing construction without allowing too much damage to the initial structure. By adding the required base layer or stabilising an existing base layer of the pavement at the initial construction phase, the bearing capacity is increased to the actual bearing capacity required.

The present worth of cost (PWOC) values for the staged construction options listed in Table 8.20 vary from being almost the same as for the full construction options to values significantly less than the PWOC for the full construction options. In general, staged construction therefore spreads the financial burden of construction and is economically more viable than initial full construction. The economics of each project must, however, be considered on merit. It must also be kept in mind that because some of the cost of construction is shifted a few years into the life cycle of a pavement, future budgets must allow for this cost plus inflation.

The details of upgrading from a gravel street to a surfaced street will depend largely on individual projects and will be determined by the bearing capacity of the material on the existing street. As already mentioned, the strength of the existing pavement should be optimally utilised in the new design and if material is imported to the gravel street,

Table 8.20: Example of staged construction

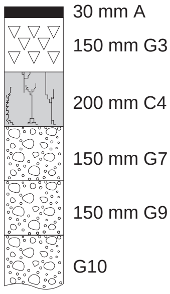
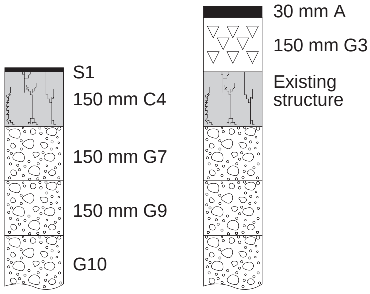
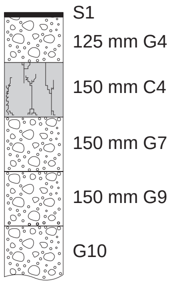
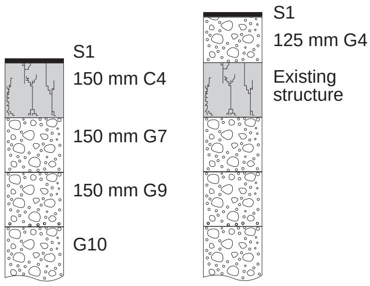
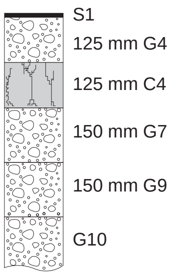
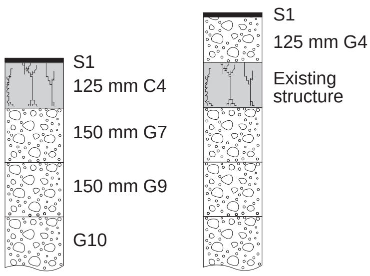
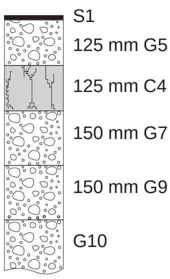
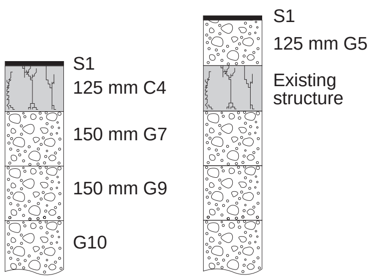
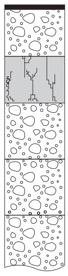
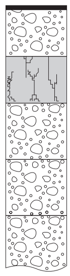
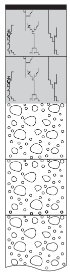
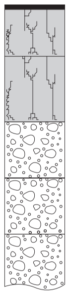
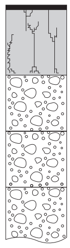
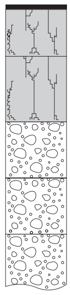
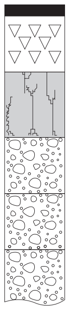
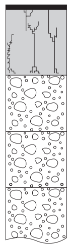
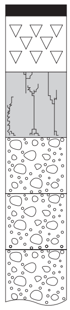
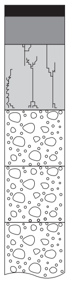
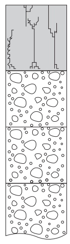
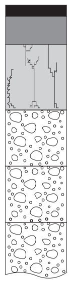
Description	Full construction	Staged construction
Initial construction: Lightly cemented base (pavement catalogue, street category UD): design bearing capacity 100 000 E80s *Final construction:* Granular base (pavement catalogue for dry regions, street category UB): design bearing capacity 3 million E80s Final pavement suitable for minor arterials, collectors and bus routes		 Addition of granular base during final construction
Initial construction: Lightly cemented base (pavement catalogue, street category UD): design bearing capacity 100 000 E80s *Final construction:* Granular base (pavement catalogue for dry regions, street category UB): design bearing capacity 1 million E80s Final pavement suitable for minor arterials, collectors and bus routes		 Addition of granular base during final construction
Initial construction: Lightly cemented base (pavement catalogue, street category UD): design bearing capacity 30 000 E80s *Final construction:* Granular base pavement (catalogue for dry regions, street category UC): design bearing capacity 1 million E80s Final pavement suitable for residential collector streets and lightly trafficked bus routes		 Addition of granular base during final construction
Initial construction: Lightly cemented base (pavement catalogue, street category UD): design bearing capacity 30 000 E80s *Final construction:* Granular base pavement (catalogue for dry regions, street category UC): design bearing capacity 300 000 E80s Final pavement suitable for residential collector streets and lightly trafficked bus routes		 Addition of granular base during final construction

Table 8.20: Example of staged construction (continued)

Description	Full construction	Staged construction
Initial construction: Granular base (pavement catalogue for dry regions, street category UC): design bearing capacity 100 000 E80s *Final construction: Lightly cemented base* (pavement catalogue, street category UC): design bearing capacity 1 million E80s Final pavement suitable for residential collector streets and lightly trafficked bus routes	 S1 125 mm G5 125 mm C4 150 mm G7 150 mm G9 G10	 S1 125 mm G5 125 mm C4 150 mm G7 150 mm G9 G10  S1 125 mm C3 Existing structure Deep in-place milling and stabilisation of granular base during final construction
Initial construction: Lightly cemented base (pavement catalogue, street category UC): design bearing capacity 300 000 E80s *Final construction:* Lightly cemented base (pavement catalogue, street category UC): design bearing capacity 3 million E80s Final pavement suitable for minor arterials, collectors and bus routes	 S1 125 mm C3 200 mm C4 150 mm G7 150 mm G9 G10	 S1 200 mm C4 150 mm G7 150 mm G9 G10  S1 125 mm C3 Existing structure Addition of lightly cemented base during final construction
Initial construction: Lightly cemented base (pavement catalogue, street category UC): design bearing capacity 300 000 E80s *Final construction:* Granular base pavement (catalogue for **wet regions**, street category UB): design bearing capacity 3 million E80s Final pavement suitable for minor arterials, collectors and bus routes in **wet regions**	 30 mm A 150 mm G1 200 mm C4 150 mm G7 150 mm G9 G10	 S1 200 mm C4 150 mm G7 150 mm G9 G10  30 mm A 150 mm G1 Existing structure Addition of a crushed stone base layer during final construction
Initial construction: Lightly cemented base (pavement catalogue, street category UC): design bearing capacity 300 000 E80s *Final construction:* Ashalt hot-mix base (pavement catalogue, street category UB): design bearing capacity 1 million E80s Final pavement suitable for minor arterials, collectors and bus routes in **wet regions**	 30 mm A 80 mm BC 200 mm C4 150 mm G7 150 mm G9 G10	 200 mm C4 150 mm G7 150 mm G9 G10  30 mm A 80 mm BC Existing structure Addition of asphalt hot-mix base layer during final construction

the possible utilisation of this material in a future upgrading to a paved street should be kept in mind.

The cost analysis for upgrading from a gravel to a surfaced street must at least include the savings in vehicle-operating cost as part of the benefit to the street user. The cost of upgrading should be weighed against the benefits by means of a cost-benefit analysis, expressing the benefits as a ratio to the cost. The CB-Roads or SURF+ (refer to manuals) computer programs are ideally suited for this type of analysis.

Construction approaches

Construction of urban streets has become a highly mechanised process but, over the past few years, the possibility of creating employment opportunities has led to greater use of labour-intensive technologies. Additionally, the encouragement of small businesses, owned by the previously disadvantaged, has led to the greater use of these affirmable business enterprises (ABEs) as subcontractors to established contractors or as contractors for small projects. The benefits of using labour-intensive construction or of using ABEs would include a reduction in unemployment, by creating productive jobs and opportunities.

There are thus three construction approaches that can be adopted, although a mix may also be appropriate:

- conventional construction (mechanised);
- labour-intensive construction; and
- construction using ABEs.

The method of construction that is to be used may have some impact on the selection of materials and the structural design. The construction method should be clearly understood before the design proceeds, as a design suitable for plant-intensive construction may be unsuitable for labour-based construction and vice versa.

Conventional construction

Conventional construction is generally well understood by engineers and most design in the recent past has been for this type of construction. Conventional construction is suited to most new street-construction jobs, perhaps with the exception of construction in confined areas. Advantages may include rapid mobilisation and completion, while disadvantages may include limited expenditure on the local community.

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Labour-based construction

If it is important to include the achievement of socio-economic goals as part of service provision, the designer should consider the use of labour-based construction to achieve these goals. Socio-economic goals that could motivate the use of labour-based construction could include relief of unemployment and the transfer of construction skills to the unemployed, particularly previously disadvantaged persons, with the aim of developing SMMEs.

It is important that labour-intensive construction be carried out using a payment-for-production, or task-based, approach. This is the only way in which economic efficiency has been realised.

Small, medium and micro-enterprises (SMMEs)

Where there is a motivation to use labour-based construction (if it is important to include the achievement of socio-economic goals as part of service provision), the designer should also consider the promotion of local contractors to achieve these goals. Socio-economic goals that could motivate the use of SMMEs could include the promotion of entrepreneurship and the advancement of local businesses, particularly those owned by previously disadvantaged persons.

DESIGNING FOR LABOUR-BASED CONSTRUCTION

In order to ensure the maximisation of job creation to the extent that is economic and feasible, the terms of reference for technical consultants engaged to carry out feasibility studies should require the consultant to examine the appropriateness of designs that are inherently labour-intensive, to report on the economic implications of using such designs and, thereafter, to design a project based on designs and technology appropriate for construction that maximises labour-intensive methods.

All construction activities cannot always be executed by means of labour-intensive methods. This must be recognised in the design. Examples of activities demanding greater mechanisation are:

- deep excavation - apart from safety considerations, material can only be thrown a certain height by shovel;
- excavation and spreading of very coarse material;
- in-situ mixing of stabilising material (cement or lime) effectively into coarse aggregates;
- application of tar - due to safety considerations;
- compaction of thick layers or very large aggregates (e.g. rock fill) with small (pedestrian) rollers;
- mixing of high-strength concrete;

- excavation of medium to hard material;
- haulage by wheelbarrows over long distances; and
- placement of heavy pipes.

Labour-intensive construction should, in general, strive to obtain the standards set for conventional construction. However, the design should ensure that the standards specified are appropriate. This necessitates a critical review of all specifications during the design stage.

Table 8.21 gives generalised information on the employment potential of roadwork projects. The information in the table is of a general nature, and should be used with caution.

The following are some guidelines that can be used in the design of civil engineering projects to maximise the use of labour:

- Once the type of project has been selected, the information in Table 8.22 can be used to indicate the potential for employment creation.
- Select the construction activities that have the biggest impact on employment creation (where the contribution of this activity forms a significant part of the project cost and the activity has the potential for employment creation). Information in the following paragraphs will be useful in this selection. A preliminary cost analysis can be done.
- Consider using local plant and materials (i.e. rent plant and purchase material from the community).
- Consider the involvement of ABEs, which generally use more labour-intensive methods, in the construction.

Table 8.21: Summary of employment potential

PROJECT	CONVENTIONAL CONTRIBUTION (%)			POTENTIAL CONTRIBUTION (%)		
	LABOUR	PLANT	MATERIAL	LABOUR	PLANT	MATERIAL
Rehabilitation*	9	53	37	29	26	45
Gravel road*	15	66	19	49	35	16
Drainage (culverts)	34	36	30	54	24	22
Bridges	20	10	70	22	8	70
Urban street	13	54	33	36	27	37

*Earthworks and pavements only.

Table 8.22: Relative contribution of main activities

DESCRIPTION	% CONTRIBUTION TOWARDS PROJECT COSTS		
	REHABILITATION	PAVED	GRAVEL
Site accommodation	3	2	4,5
Accommodation of traffic	5	5	4
Clearing and grubbing	0,5	1	2
Drainage	3	3	7,5
Culverts	3	15	11
Kerbs and edging	3,5	8,5	0
Earthworks	6	4	22
Pavement layers	10	14	16
Base	8	10	0
Prime and seal work	35	15	0
Ancillary works	5	4	6,5
Landscaping	2	1	

- Conduct a detailed investigation into the activities selected for labour-intensive construction, and the possibilities for using ABEs, local plant and materials. The result of this investigation may indicate that some activities cannot be done by means of labour, due to construction practicalities or the availability of materials. A more detailed cost analysis can then be done. Further, make sure that the design can be specified.

  Tables 8.23 and 8.24 may be of assistance for determining the most appropriate activities to be undertaken by labour-intensive methods. The contribution of the various elements towards the contract value and the potential of the various elements for employment enhancement are given.

Construction using ABEs

ABEs that are interested in carrying out some of the project work should be identified. A certain amount of technical and business training may be required for inexperienced ABEs.

In the case of street construction projects, the use of ABEs is likely to include

- emerging small contractors;
- emerging materials suppliers; and
- emerging materials hauliers.

The activities in which ABEs may be engaged can include all construction operations, including the entire works for small projects. Once the ABEs who might wish to work on a project have been identified, the technical consultant should propose,

Table 8.23: Potential of pavement layers for labour-intensive construction methods

LAYER	TYPE	POTENTIAL
Subbase	In-situ soil	Good
	Imported	Good*
	Stabilised soil	Fair, not practical
Base	In-situ soil	Good
	Natural gravel	Good
	Emulsion treated gravel	Good
	Crusher run	Fair, not practical
	Cement stabilised gravel	Not practical**
	Lime stabilised gravel	Fair, good***
	Bituminous premix	Fair, good
	Waterbound macadam	Good
	Penetration macadam	Good
Surfacing	Sand seal	Good#
	Slurry	Good
	Double seal	Good#
	Cape seal	Good#
	Asphalt	Fair
	Roller compacted concrete	Good
	Concrete (plain)	Good
	Concrete (reinforced)	Good
	Segmental blocks	Good##

Notes:

* The suitability of this will depend entirely on the haul distance.

** Cement-stabilised gravel is not suitable for labour intensive methods due to its quick setting time.

*** Lime-stabilised gravel is more suitable as it reacts and sets more slowly, but achieving an even mix is difficult by entirely manual means and labourers must take extreme care to avoid contact between the lime and skin during application: protective clothing is essential.

In the case of a bitumen surfacing, only certain types of emulsion have a non-critical application temperature and are suitable for hand laying.

Quality control of on-site manufacture is critical.

and obtain agreement to, the procurement format that is best suited to the size and scope of the project and the capabilities of the ABEs. He may need to identify packages of work that can be carried out by ABEs, in which case the tender documents for the main contractor should specifically identify such packages of work. Typically, ABEs have been used in the activities shown in Table 8.24 .

MAINTENANCE

The object of maintenance is to preserve, repair and restore the maintainable features of a street network to their designed standard or to a predetermined condition. In this sense, even rehabilitation may be considered a maintenance activity as the object would be to restore the pavement to its original condition. Proper, timely maintenance that is managed efficiently will result in long-term savings to the street authority and street user.

In addition to normal street maintenance activities, the situation in densely populated areas is further complicated by the installation and maintenance of all the other services found in the street reserve in such areas. These services include stormwater, electricity and water supply, sewerage and telecommunication systems. Although the street authority is not responsible for the maintenance of these systems, it remains the legal custodian of the street reserve and

Table 8.24: Typical activities suited to ABEs

COMPONENT	ACTIVITIES
Accommodation of traffic	Watering of gravel diversions
Clearing and grubbing	As required
Drainage	Catchpits and manholes Excavation of open drains Lined open drain Subsoil drains
Culverts	Inlet and outlet structures Excavation of trenches Installation of lightweight pipes Manufacture of reinforced concrete slabs, walls and decks Masonry walls
Kerbing and edging	Manufacture of concrete elements Laying of kerbing and edging
Earthworks	Minor earthworks
Pavement layers	Crushing of aggregates Screening of stockpiles Haulage of materials Spreading of materials Removal of oversize materials
Base	Construction of labour-intensive base types (waterbound macadam, emulsion-treated base, stabilised or unstabilised gravel) Manufacture of paving blocks Laying of paving blocks
Prime and seal work	Hand spraying of prime Manufacture and laying of slurry Seals (Cape seal, double seal, single seal)
Ancillary works Masonry walls	Fencing Gabions Concrete structures Painting of roadmarkings
Landscaping	Grassing Planting of trees

as such must authorise and coordinate the efforts of the various other departments or companies involved. Complaints regarding the poor or delayed repair of the street surface operations of other parties will more often than not be directed at the streets department, and these activities should therefore be monitored by the streets department.

The two main components of the maintenance process are the actual physical execution of the work and the management of the process. Of these two, the biggest challenge is the efficient management of the process. Management systems may vary from being highly sophisticated systems of a pro-active nature with a large computerised component, to less technical needs-driven systems of a reactive nature, or even a combination of the two. The success of a particular system largely depends on the environment in which it is applied and may well fail if it is not appropriate to the specific conditions and demands of the area where it is applied and the type of maintenance activity to which it is applied.

Maintenance activities may be classified as being routine or periodic maintenance, or rehabilitation. Within these categories, some activities may be specific to paved streets and some to unpaved streets, while others may be applicable to both street types. By defining the scope of the maintenance for which a particular authority will be responsible, in terms of the maintenance activities applicable to the particular street network, a better understanding of exactly what is required is achieved. Table 8.25 lists some maintenance activities according to maintenance category.

Some estimate of the extent of the maintenance workload will set the scene for planning, executing and monitoring maintenance activities. This may be done by dividing the whole network into maintenance zones and preparing an inventory, indicating the expected amount of work for each maintenance activity in each zone. For instance, the length of unpaved street will directly determine the workload in terms of watering and blading for a particular zone.

Once the scope and extent of maintenance are known, several options may be considered for deciding on when to do a specific activity and on the best means of doing the work.

The simplest approach to routine maintenance activities is to work according to predetermined schedules, repeated at regular intervals. Maintenance teams will therefore be set up to perform one or more maintenance activities and will be allocated a specific maintenance zone. They will then work through the zone according to a predetermined schedule and will repeat the schedule in cycles. The resources should be balanced with the maintenance workload in the maintenance zones, to control the duration of the cycles. If these cycles are too short, the teams will be under-utilised and if too long, maintenance problems will be left unattended to for lengthy periods.

Typically, the cycles could have a duration of a few months for activities such as vegetation control to a few years for painting street markings. The planning horizon is determined by the duration of these cycles. Actual performance may be measured against the set schedules and planning revised accordingly. This approach implies that the maintenance team is responsible for identifying the need for maintenance as it works through its area of responsibility. The approach may also fail to utilise the maintenance teams to full capacity and may be expensive as the whole team is moved through the maintenance zone regardless of the need for maintenance. It does, however, remain a fairly straightforward, unsophisticated approach.

Table 8.25: Street maintenance categories and limited examples of maintenance activities

FACILITY	MAINTENANCE CATEGORY		
	ROUTINE	PERIODIC	REHABILITATION
Paved streets	• Painting street markings • Cleaning stormwater inlets • Pothole patching • Crack sealing • Sidewalk and cycle path repair.	• Reseal	• Recycle • Overlay • Recycle and overlay
Unpaved streets	• Watering • Blading • Patch gravelling • Cleaning side drains and ditches	• Regravel	• Shape and regravel
Non-specific	• Street sign maintenance • Vegetation control	• Street sign replacement	

The alternative is to have inspectors identifying the need for maintenance at specific locations and then targeting maintenance teams at those problems with a planning horizon of about two weeks.

Periodic maintenance and rehabilitation are normally triggered by the street condition. In these cases, the process may be initiated by a pro-active system, based on a history of condition-assessment data collected over years, processed by computer and enhanced by the experience of the persons involved in the management system, or by a reactive system where areas requiring immediate action are identified and attended to. If the latter approach is selected, a rapid response is, however, required to prevent excessive deterioration. In addition to the normal working procedure opted for, a complaints register should also be kept. Prompt reaction to the complaints raised by the public in such a register is, however, essential.

Maintenance activities may be done by in-house maintenance teams or given out on contract. The recent trend is towards appointing contractors to do maintenance. Because of the low level of technical expertise required - especially by routine maintenance activities - this offers an excellent entry point for aspiring future contractors. Maintenance activities are also ideally suited to manual labour. Proper training and logistical support of the maintenance teams are, however, essential and must be specifically targeted at the activities they will perform.

The lack of logistical support in terms of transport, hand tools and materials may seriously handicap maintenance efforts. On the other hand, over- supply, by keeping too much stock of an item for which there is a low demand, will also tie up capital. This situation may easily arise when there is too much diversity in, for instance, the selection of paving blocks used. Stocks of a particular block will have to be acquired and stored for replacing broken blocks in future. If there is too wide a selection of blocks used, stocks will have to be obtained and stored for each type, resulting in a major logistical and financial problem. It is therefore advisable to standardise on certain bulk items.

Maintenance activities are in essence simple procedures to be carried out with the proper training and equipment. Management during all stages - including planning, execution, monitoring and re-planning, as well as addressing the vast logistical demands of maintenance teams - may at the end of the day determine the success of the maintenance efforts.

MAINTENANCE OF BASIC ACCESS STREETS

Maintenance funds for basic access streets constructed with severe budgetary constraints are likely to be limited. Local authority resources or experience may also not be able to meet the need. Because of the light traffic and possible limitations in the maintenance applied, basic access streets are designed to withstand environmental deterioration. Under such a policy, maintenance of drainage channels is more important than grading unsurfaced streets. The emphasis is on labour-intensive types of maintenance rather than equipment.

Labour and mechanisation

Some construction and most maintenance activities offer considerable scope for the application of labour-based methods, and some activities are only possible by such methods. Table 8.26 indicates the potential for mechanical and labour-based methods in different maintenance operations. In choosing between mechanical and labour-based methods, consideration should be given to the standard of work achieved by each method, as well as to costs and the organisation of work. On basic access streets it is not always necessary for labour-based operations to have the same standard of finish that can be obtained using machines. A decision to do labour-based maintenance may influence design details such as the shape of side drainage channels.

Environmental maintenance

Erosion is the main form of wear in basic access streets. The maintenance of such streets is therefore primarily related to the environmental maintenance of the drainage facilities. This includes the shaping, cleaning, desilting and scour protection of the drainage channels. Grass-cutting and other forms of vegetation control will also constitute part of this type of maintenance. The emphasis is on the efficient functioning of drainage channels.

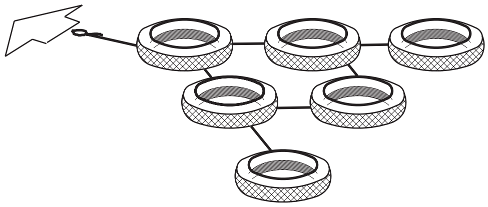
The integrity of a surfacing on light-structure streets determines the lifespan of a street. Crack-sealing or pothole-patching should be done on a preventive basis, as cracked or potholed streets may show rapid deterioration when structural layers are wet and carrying traffic.

MAINTENANCE OF TERTIARY WAYS

The construction of tertiary ways is mostly labour-intensive and maintenance should be as well. This type of maintenance can be classified as environmental maintenance, as most would be focused on ditch-cleaning, grass-cutting, scour protection and replacing gravel. Various labour-intensive means exist to enable the scraping of tertiary ways. Figure 8.21 illustrates typical drags that can be used for this purpose.

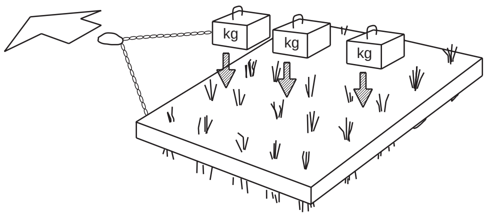
BRUSHWOOD DRAG

Small branches tied together



TYRE SLEDGE

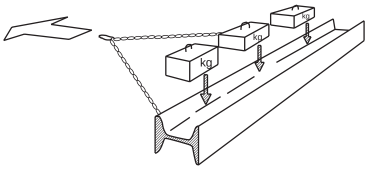
Old truck tyres chained together



CABLE DRAG

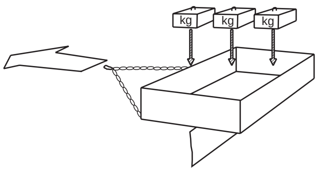
Bundles of steel cables bound together and fixed in a frame, weighed with concrete blocks to enable it to cut into the surface. Wooden sticks may be used if steel cables are not available.

Care must be taken that pieces of the steel cable which may break off the drag are not left lying on the road.



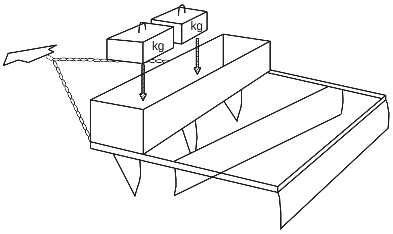
STEEL DRAG

Rolled steel joist or steel rail weighted with concrete blocks and towed at an angle to the road.



BOX DRAG

Wooden box filled with concrete blocks on top of an old grader blade.



TOLARD

Five blades at different angles under a box weighted with concrete blocks.

Figure 8.21: Simple drags for maintenance of tertiary ways

Table 8.26: Suitability of mechanical equipment and labour for maintenance activities

ACTIVITY	MECHANICAL EQUIPMENT	LABOUR
Ditch-cleaning and cutting	Good*	Good*
Cleaning and minor repair to culverts and bridges	Poor	Good
Building scour controls	Poor	Good
Repair of structures	Poor	Good
Grading unpaved surfaces	Good	Impracticable
Dragging and brushing of unpaved surfaces	Poor	Poor
Filling potholes	Poor	Good
Filling unpaved surfaces and slopes	Poor	Good
Grass cutting	Good**	Good
Repairing and replacing traffic signs	Poor	Good
Stockpiling gravel	Good	Fair
Regravelling gravel surfaces	Good	Fair

* The labour potential in these activities depends on suitable design of the ditch cross-section (see Figure 8.5).

** The labour potential in this activity depends on the width of the shoulder and the presence of obstructions, such as road furniture and culvert headwalls.

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Abbreviations:

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CSRA:	Committee of State Road Authorities, South Africa
CUTA:	Committee of Urban Transport Authorities, South Africa
HRB:	Highway Research Board, Washington
KZN:	KwaZulu-Natal Department of Transport, Pietermaritzburg
NITRR:	National Institute for Transport and Road Research, CSIR, Pretoria

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APPENDIX A

THE CATALOGUE OF PAVEMENT DESIGNS

The catalogue of pavement designs contained in this appendix was compiled from various sources, with the aim of addressing the specific needs of pavement design in urban areas. The catalogue pages dealing with the granular base pavements for both wet and dry regions, the cement-treated base pavements, and the asphalt hot-mix base pavements were obtained from the TRH4 document. These designs were developed from the SAMDM and were checked with the DCP design method.

The macadam page was obtained form the TRH4 document and supplemented by the category D designs.

The emulsion-treated base pavement designs were developed using the DCP design method.

The block paving pavement designs were obtained from the UTG2 document.

The cast-in-situ block paving designs were obtained from the original proposal by Visser (1994), which was subsequently validated by Visser and Hall (1999).

Notes regarding the use of the catalogue

- The designs in the catalogue are merely suggested designs, and the designer may modify and adjust them on the basis of sound engineering principles and experience.
- The layer thicknesses indicated in the catalogue represent minimum thicknesses required. Practical construction issues and layer thickness tolerances should be allowed for.
- Although the catalogue generally indicates a fixed thickness of asphalt, regardless of the grading of the asphalt and the binder type, these values serve only as suggested starting values. The designer should consider the particular properties of the asphalt mix that will be used.
- Designs are valid for the upper bearing-capacity boundaries of the pavement classes.
- It is assumed that all the requirements regarding material quality, moisture condition and compaction are met during the construction of the pavements.

RECOMMENDED READING

CSRA (1996). *Structural design of flexible pavements for inter-urban and rural roads.* Department of Transport Technical Recommendations for Highways Draft TRH4. Pretoria.

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GRANULAR BASES (DRY AND MODERATE REGIONS)

1998

ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										
	ES0,003 $0{,}1\text{-}0{,}3\times10^4$	ES0,01 $0{,}3\text{-}1{,}0\times10^4$	ES0,03 $1{,}0\text{-}3{,}0\times10^4$	ES0,1 $3{,}0\text{-}10\times10^4$	ES0,3 $0{,}1\text{-}0{,}3\times10^6$	ES1 $0{,}3\text{-}1{,}0\times10^6$	ES3 $1{,}0\text{-}3{,}0\times10^6$	ES10 $3{,}0\text{-}10\times10^6$	ES30 $10\text{-}30\times10^6$	ES100 $30\text{-}100\times10^6$	Foundation
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes							40A 125 G2 150 C3 40A 150 G2 150 G5	40A 150 G2 250 C3	50A 150 G1 250 C3	50A 150 G1 300 C3	
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-areas and bus routes						S 125 G4 150 C4 S 150 G4 150 G5	S*/30A 150 G3 150 C4 S*/30A 150 G3 150 G5	40A 150 G2 200 C4 30A 150 G2 200 G5			150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes				S 100 G5 125 C4 S 125 G4 125 G6	S 125 G5 125 C4 S 125 G4 150 G6	S 125 G4 125 C4 S 125 G4 150 G5	S 150 G3 150 C4 S 150 G3 150 G5				
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac	S1 100 G5 100 G7	S1 100 G5 125 G7	S1 100 G4 125 G7	S1 100 G4 125 G6 S1 100 G5 100 C4	S 125 G4 125 G6 S 100 G5 125 C4	S 125 G4 150 G6 S 125 G5 150 C4					150 G9 G10

Most likely combinations of road category and design bearing capacity.

Symbol A denotes AG, AC, or AS.
A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment
* If seal is used, increase C4 and G5 subbase thickness to 200mm.

GRANULAR BASES (WET REGIONS)

1998

ROAD CATEGORY	ES0,003 $0{,}1\text{-}0{,}3\times10^4$	ES0,01 $0{,}3\text{-}1{,}0\times10^4$	ES0,03 $1{,}0\text{-}3{,}0\times10^4$	ES0,1 $3{,}0\text{-}10\times10^4$	ES0,3 $0{,}1\text{-}0{,}3\times10^6$	ES1 $0{,}3\text{-}1{,}0\times10^6$	ES3 $1{,}0\text{-}3{,}0\times10^6$	ES10 $3{,}0\text{-}10\times10^6$	ES30 $10\text{-}30\times10^6$	ES100 $30\text{-}100\times10^6$	Foundation
	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes							30A 150 G1** 200 C3	40A 150 G1 300 C3 *(250 C3)	50A 150 G1 400 C3 *(300 C3)		150 G7 150 G9 G10
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-areas and bus routes						S 150 G2 150 C4 S 150 G2 200 G5	S/30A 150 G1** 200 C4	40A 150 G1 300 C4 *(250 C4)			
UC: Residential access collectors, car parks and lightly trafficked bus routes				S 100 G5 125 C4 S 125 G4 125 G6	S 125 G5 125 C4 S 150 G4 150 G6	S 125 G2 150 C4 S 150 G2 150 G5	S 150 G2** 200 C4 S 150 G2 150 G4				
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac	S1 100 G5 100 G7	S1 100 G5 125 G7	S1 100 G4 125 G7	S1 100 G4 125 G6 S1 100 G5 100 C4	S 125 G4 125 G6 S 100 G5 125 C4	S 150 G4 150 G6 S 125 G5 150 C4					150 G9 G10

Most likely combinations of road category and design bearing capacity.

Symbol A denotes AG, AC, or AS.
A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment
* If water is prevented from entering the base, the subbase thickness may be reduced to the values indicated in brackets.
** Base thickness may be reduced by 25 mm if cemented subbase thickness is increased by 50 mm.

CEMENTED BASES

ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										
	ES0,003 0,1-0,3x10^4	ES0,01 0,3-1,0x10^4	ES0,03 1,0-3,0x10^4	ES0,1 3,0-10x10^4	ES0,3 0,1-0,3x10^6	ES1 0,3-1,0x10^6	ES3 1,0-3,0x10^6	ES10 3,0-10x10^6	ES30 10-30x10^6	ES100 30-100x10^6	Foundation
UA: Trunk roads, primary distributors, freeways, arterials and by-passes							30A 150 C3 200 C4				
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-areas and bus routes						S 125 C3 150 C4	S 125 C3 200 C4	S/30A 150 C3* 300 C4			150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes				S 100 C4 100 G6	S 200 C3 S 125 C4 125 G6	S 125 C3 125 C4	S 150 C3 150 C4				
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac	S1 100 C4 100 G8	S1 100 C4 125 G8	S1 125 C4 125 G7	S1 150 C4 150 G7	S 125 C4 125 G6	S 125 C4 150 G6					150 G9 G10

Symbol A denotes AG, AC, or AS.
A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment
* Crushing of cemented base may occur

Most likely combinations of road category and design bearing capacity.

1998

ASPHALT HOT-MIX BASES

ROAD CATEGORY	ES0,003 0,1-0,3x10 4	ES0,01 0,3-1,0x10 4	ES0,03 1,0-3,0x10 4	ES0,1 3,0-10x10 4	ES0,3 0,1-0,3x10 6	ES1 0,3-1,0x10 6	ES3 1,0-3,0x10 6	ES10 3,0-10x10 6	ES30 10-30x10 6	ES100 30-100x10 6	Foundation
	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes							40A 80 BC 250 C3	40A 90 BC 300 C3	40A 120 BC 400 C3	50A 180 BC 450 C3	
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-loading areas and bus routes							30A 80 BC 200 C4	30A 80 BC 300 C3			150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes											
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac											150 G9 G10

Symbol A denotes AG, AC, or AS.
A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray
Symbol BC does not include LAMBS (BC1 Table 13)
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment

Most likely combinations of road category and design bearing capacity.

EMULSION-TREATED BASES

1998

ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE) ES0,003 $0,1\text{-}0,3x10^4$	ES0,01 $0,3\text{-}1,0x10^4$	ES0,03 $1,0\text{-}3,0x10^4$	ES0,1 $3,0\text{-}10x10^4$	ES0,3 $0,1\text{-}0,3x10^6$	ES1 $0,3\text{-}1,0x10^6$	ES3 $1,0\text{-}3,0x10^6$	ES10 $3,0\text{-}10x10^6$	ES30 $10\text{-}30x10^6$	ES100 $30\text{-}100x10^6$	Foundation
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes								40A 125 E1 250 C4			
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-loading areas and bus routes						S 100 E2 150 C4 S 125 E2 150 G5	S*/30A 125 E2 150 C4 S*/30A 150 E2 150 G5	40A 125 E1 200 C4 40A 150 E1 150 C4			150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes				S 100 E3 150 G6	S 100 E2 150 G6	S 100 E2 125 C4					
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac			S1 100 E3 100 G6	S1 100 E3 125 G6							150 G9 G10

Symbol A denotes AG, AC, or AS.
A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment

Most likely combinations of road category and design bearing capacity.

MACADAM BASES

1998

ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE) ES0,003 0,1-0,3x10^4	ES0,01 0,3-1,0x10^4	ES0,03 1,0-3,0x10^4	ES0,1 3,0-10x10^4	ES0,3 0,1-0,3x10^6	ES1 0,3-1,0x10^6	ES3 1,0-3,0x10^6	ES10 3,0-10x10^6	ES30 10-30x10^6	ES100 30-100x10^6	Foundation
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes							30 - 40A 125 M 150 C3	40A 150 M 125 C3 125 C4	50A 150 M 150 C3 150 C3		
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-loading areas and bus routes					S 100 M 150 G5	S 125 M 150 G5	S or 30A 125 M 150 C4	40A 125 M 125 C4 125 C4			150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes				S 100 M 100 C4 S 100 M 125 G5	S 100 M 125 C4	S 100 M 150 G5	S 125 M 100 C4 S 125 M 150 G5				
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac	S1 100 M 150 G7 S1 100 M 125 C4	S1 125 M 150 G7 S1 100 M 125 C4	S1 125 M 150 G7 S1 75 M 125 C4	S1 150 M 150 G7 75 M 125 C4 150 G7	S 100 M 125 C4 150 G7						150 G9 G10

Symbol A denotes AG, AC, or AS.
A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment

Most likely combinations of road category and design bearing capacity.

1998

BLOCK PAVEMENTS (MODERATE AND DRY REGIONS)

ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										Foundation
	ES0,003 $0,1\text{-}0,3\times10^{4}$	ES0,01 $0,3\text{-}1,0\times10^{4}$	ES0,03 $1,0\text{-}3,0\times10^{4}$	ES0,1 $3,0\text{-}10\times10^{4}$	ES0,3 $0,1\text{-}0,3\times10^{6}$	ES1 $0,3\text{-}1,0\times10^{6}$	ES3 $1,0\text{-}3,0\times10^{6}$	ES10 $3,0\text{-}10\times10^{6}$	ES30 $10\text{-}30\times10^{6}$	ES100 $30\text{-}100\times10^{6}$	
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes											
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-loading areas and bus routes						60 S-A S-B or S-C 20 SND 125 C4 60 S-A S-B or S-C 20 SND 100 - 150 G5	60 S-A S-B or S-C 20 SND 150 C4 60 S-A S-B or S-C 20 SND 150 G5	80 S-A 20 SND 100 C4 100 C4	80 S-A 20 SND 125 C3 125 C3		150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes				60 S-A S-B or S-C 20 SND 100 - 125 G5		60 S-A S-B or S-C 20 SND 125 C4 60 S-A S-B or S-C 20 SND 100 - 125 G5	60 S-A S-B or S-C 20 SND 125 C4 60 S-A S-B or S-C 20 SND 150 G5	60 S-A S-B or S-C 20 SND 125 C4 125 C4	80 S-A S-B or S-C 20 SND 150 C3 150 C4		
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac			60 S-A S-B or S-C 20 SND								150 G9 G10

(shaded) Most likely combinations of road category and design bearing capacity.

BLOCK PAVEMENTS (WET REGIONS)

1998

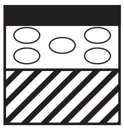
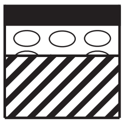
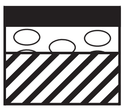
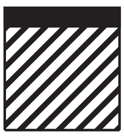
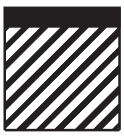
ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										Foundation
	ES0,003 0,1-0,3x10^4	ES0,01 0,3-1,0x10^4	ES0,03 1,0-3,0x10^4	ES0,1 3,0-10x10^4	ES0,3 0,1-0,3x10^6	ES1 0,3-1,0x10^6	ES3 1,0-3,0x10^6	ES10 3,0-10x10^6	ES30 10-30x10^6	ES100 30-100x10^6	
UA: Trunk roads, primary distributors, freeways, major arterials and by-passes											
UB: District and local distributors, minor arterials and collectors, industrial roads, goods-areas and bus routes						60 S-A S-B or S-C 20 SND 150 G5 60 S-A S-B or S-C 20 SND 150 C4	60 S-A S-B or S-C 20 SND 150 C4	60-80 S-A 20 SND 125 C4 125 C4	80 S-A 20 SND 150 C4 150 C4		150 G7 150 G9 G10
UC: Residential access collectors, car parks and lightly trafficked bus routes				60 S-A S-B or S-C 20 SND 100 - 125 G5 60 S-A S-B or S-C 20 SND 100 C4		60 S-A S-B or S-C 20 SND 100- 150 G5 60 S-A S-B or S-C 20 SND 100- 150 C4	60 S-A S-B or S-C 20 SND 150 G5 60 S-A S-B or S-C 20 SND 125 C4				
UD: Local access roads, -loops, -ways, -courts, -strips and culs de sac			60 S-A S-B or S-C 20 SND								150 G9 G10

Most likely combinations of road category and design bearing capacity.

Table A1: Cast in-situ block pavement codes for use in Figure A1

ROUTE AND ENVIRONMENT CLASSIFICATION	DESIGN TRAFFIC DURING STRUCTURAL LIFE (E80)		
	$<0,2 \times 10^6$	$0,2\text{-}0,8 \times 10^6$	$0,8\text{-}3,0 \times 10^6$
Minor arterial, wet climate, CBR 3-7*		A	A
Minor arterial, wet climate, CBR >7		A	A
Minor arterial, dry climate, CBR 3-7		B	B
Minor arterial, dry climate, CBR >7		C	B
Access street, wet climate, CBR 3-7	D	C	B
Access street, wet climate, CBR >7	E	C	B
Access street, dry climate, CBR 3-7	D	C	B
Access street, dry climate, CBR >7	E	C	B

*Special precautions have to be taken if the design CBR <3

Pavement code	Pavement structure	
A		75/30 MPa cell slab 150 G5 or 150 C4 In-situ*
B		75/30 MPa cell slab 150 G5 or 125 C4 In-situ*
C		75/30 MPa cell slab 125 G5 or 100 C4 In-situ*
D		75/20 MPa cell slab In-situ*
E		50/20 MPa cell slab In-situ*

Note: The strength of the cell slab indicated is the 28 day cube strength
* Special precaution have to be taken if the design CBR <3

Figure A1: Catalogue of structural designs for cost in-situ block pavements

APPENDIX B

EXAMPLES OF STRUCTURAL DESIGN BY THE CATALOGUE METHOD

A. EXAMPLE OF THE STRUCTURAL DESIGN OF A CATEGORY UB STREET

SERVICE OBJECTIVE, STREET CHARACTERISTICS AND STREET PROFILE

The structural design of the pavements for a new residential area needs to be done. The layout plan and stormwater design have been completed. For the purpose of this example, the design of one of the streets will be considered. The following general information is available:

- The pavement structure for a local distributor that will serve as a bus route needs to be designed. The facility must open for traffic in five years time.

- The route is regarded as an important link in the transportation system.

- Drainage will consist of a pipe system with kerbs and gutters. The pipe system is installed to avoid any future disruption on the bus route due to upgrading the drainage system.

- The traffic is expected to exceed 700 vehicles per day with 15% being heavy vehicles, mostly buses. The expected annual average daily equivalent (AADE) design traffic is 180 E80s/lane/day for this route. (If the traffic information is not readily available from the planning office (or client body), consult and follow the guidelines set out in the section on levels of service in this document).

- The street surface will be paved because of the importance of the road and the volume of traffic.

- A paved walkway and bus stops will be provided for pedestrian traffic, with controlled pedestrian crossings at intersections.

Design pavement structures of different base types and compare these on the basis of present worth of life-cycle costs before making a final selection.

Street category:
After consideration of the characteristics listed and consultation with the client, it is decided to design for a category UB street.

Design strategy

Select analysis period

The alignment will probably not change and therefore a period of 30 years is selected.

Select structural design period

As this is a new facility and there is considerable uncertainty about the traffic volume and growth rate, a period of 15 years is selected. Therefore AP = 30 years; SDP = 15 years.

Estimate design traffic

The current equivalent traffic (180 E80s/lane/day) is projected to the initial year using Equation 8.3 and the growth factor, g, from Table 8.8. The cumulative equivalent traffic over the structural design period (Equation 8.5) is determined by multiplying the initial equivalent traffic by the cumulative growth factor, f, from Table 8.9. Because of the uncertainty regarding the traffic growth rate, a sensitivity analysis with growth rates ranging from 2 to 8 percent (higher values are unrealistic for this road) is done. Table B1 shows the cumulative equivalent traffic and the applicable traffic class for a structural design period of 15 years. Regardless of the selected growth rate, the design traffic class is ES3.

Materials

Table B2 indicates the availability and cost of material. The unit prices listed are all 1995 prices.

Table B1: Cumulative equivalent traffic and applicable traffic class for SDP = 15 years

GROWTH RATE (%)	g (TABLE 8.8)	f (TABLE 8.9)	CUMULATIVE EQUIVALENT TRAFFIC (E80s) OVER SDP	DESIGN TRAFFIC CLASS (TABLE 8.5)
2	1,10	6 440	$1{,}3 \times 10^6$	ES3
4	1,22	7 600	$1{,}7 \times 10^6$	ES3
6	1,34	9 010	$2{,}2 \times 10^6$	ES3
8	1,47	10 700	$2{,}8 \times 10^6$	ES3

Table B2: Checklist of material availability and cost (base year 1995)

Symbol	Code	Material	Availability	Unit cost/m^3	
				Borrow pit	Commercial
	G1	Graded crushed stone	✔	R86,00	R122,00
	G2	Graded crushed stone	✔	R86,00	R122,00
	G3	Graded crushed stone	✔	R78,00	R116,00
	G4	Natural gravel	✔	R42,59	R57,64
	G5	Natural gravel	✔	R39,23	R53,50
	G6	Natural gravel	✔	R35,31	R50,36
	G7	Gravel/soil	✔	R33,03	R45,65
	G8	Gravel/soil	✔	R32,31	R44,87
	G9	Gravel/soil	✔	R31,40	R44,00
	G10	Gravel/soil	✗	-	-
	C1	Cemented crushed stone or gravel	✔	R106,05	R145,00
	C2	Cemented crushed stone or gravel	✗	R102,00	R140,00
	C3	Cemented natural gravel	✔	R63,00	R76,70
	C4	Cemented natural gravel	✔	R61,70	R71,70
	BEM	Bitumen emulsion modified gravel	✗	-	
	BES	Bitumen emulsion stabilised gravel	✗	-	
	BC1	Hot -mix asphalt	✔	-	
	BC2	Hot -mix asphalt	✔	R320,00	
	BC3	Hot -mix asphalt	✔	-	
	BS	Hot -mix asphalt	✔	-	
	AG	Asphalt surfacing	✔	-	
	AC	Asphalt surfacing	✔	R350,00	
	AS	Asphalt surfacing	✔	R430,00	
	AO	Asphalt surfacing	✗	-	
	AP	Asphalt surfacing	✗	-	
	S1	Surface treatment	✔	R5,10/m^2	
	S2	Surface treatment	✔	R6,00/m^2	
	S3	Sand seal	✗	R2,35/m^2	
	S4	Cape seal	✗	R6,90/m^2	
	S5	Slurry	✗	R561,00/m^3	
	S6	Slurry	✗	-	
	S7	Slurry	✗	R616,00/m^3	
	S8	Surface renewal (30%)	✗	R1,20/litre	
	S9	Surface renewal (60%)	✗	R2,03/litre	
	WM1	Waterbound macadam	✗	R150,00/m^3	
	WM2	Surface treatment	✗	R150,00/m^3	
	PM	Penetration macadam	✗	-	
	DR	Dumprock	✗	R30,00/m^3	

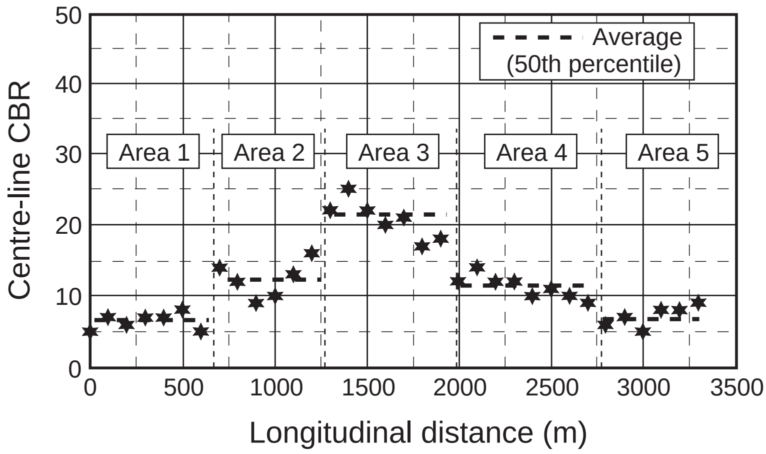


Figure B1: Centre-line CBR-values for the examples

Climatic region

The road lies within a moderate climatic region.

Delineation of subgrade areas and design CBR of subgrade

Centreline subgrade CBR values taken at 100m intervals:

5;7;6;7;7;8;5;14;12;9;10;13;16;22;25;22;20;21;17;18;12;14;12;12;10;11;10;9;6;7;5;8;8;9.

The given CBR values are illustrated in Figure B1. Visual inspection of the CBR values reveals five subgrade areas:
Subgrade area 1 : CBR = 5;7;6;7;7;8;5
Subgrade area 2 : CBR = 14;12;9;10;13;16
Subgrade area 3 : CBR = 22;25;22;20;21;17;18
Subgrade area 4 : CBR = 12;14;12;12;10;11;10;9
Subgrade area 5 : CBR = 6;7;5;8;8;9.

Table B3 gives the CBR percentile values and design CBR of the subgrade. Ten percent of the measured CBR values are allowed to be below the minimum specified for a particular material quality, and the design CBR of the subgrade is therefore determined by the 10^{th} percentile value of the measured CBR values.

Structural design and pavement type selection

Pavement type selection

From the section on materials and Table 8.17 it follows that a granular, cemented or asphalt hot-mix base layer may be used with cemented subbase layers recommended for the cemented and hot-mix base layers.

Table B3: Design CBR of the subgrade in example

SUBGRADE AREA	PERCENTILE VALUES*		CBR CLASS	DESIGN CBR OF SUBGRADE
	50 %	10 %		
1	6,2	4,4	SG3	3 - 7
2	12,0	8,8	SG2	7 - 15
3	20,5	16,7	SG1	> 15
4	11,0	8,6	SG2	7 - 15
5	7,0	4,6	SG3	3 - 7

Selected layers

The selected layers necessary for the different subgrade areas are given in Table B4 (according to Table 8.20).

Possible pavement structures

Table B5 shows possible pavement structures according to the catalogue (category UB streets, class ES3 traffic). Using a surface treatment instead of the asphalt wearing course may result in a lower riding quality. As this is an urban road with relatively low speed restrictions, this should not be a problem. A double seal is therefore used for all the designs except the hot-mix asphalt base.

Practical considerations

The designer should consider the consequences, if any, of the practical considerations given in the sections on street categories and materials.

Cost analysis

Construction cost

For the comparison of different pavement options, the construction cost of only the subbase, base and surfacing need be considered. These costs follow directly from the unit costs given in Table B2.

Future maintenance and life-cycle strategy

The structural design period is 15 years and the analysis period 30 years. The future maintenance for each pavement type can be estimated from Tables 8.16 and 8.17. Table B6 shows estimated maintenance measures during the life cycles of the different pavement options.

Discount rate

For the purposes of this example, a discount rate of 8% is normally selected. However, it is better to do a sensitivity analysis with discount rates of 6, 8 and 10%.

Salvage value and road-user costs

Although the salvage value and road-user costs may vary with pavement life, it is unlikely that the relative effect will influence the present worth of costs significantly. This can be confirmed with trial values.

Present worth of costs

The present worth of costs can be calculated from Equation 8.10. Table B7 shows the 1995 present worth of costs for each pavement type, taking the construction cost and maintenance strategy from Table B6 into account. The sensitivity analysis shows that the discount rate can vary from 6 to 10 percent without having a significant influence on the present worth of costs.

It also follows from Table B7 that the pavements with granular bases are significantly less costly than the others. Therefore, a pavement with a granular base will be selected. Considering the behaviour of the different pavement types (refer to the relevant section in this document) it is proposed that a structure with a granular base and a cemented subbase should be used. Such a pavement will probably not show the cracking and crushing problems of a pavement with a cemented base.

Table B4: Selected layers for the different subgrade areas

SUBGRADE AREA	DESIGN CBR	LOWER SELECTED LAYER	UPPER SELECTED LAYER	TREATMENT OF IN-SITU SUBGRADE
1	3 - 7	150 mm G9	150 mm G7	R+R* to 150 mm G10
2	7 - 15	-	150 mm G7	R+R to 150 mm G9
3	> 15	-	-	R+R to 150 mm G7
4	7 - 15	-	150 mm G7	R+R to 150 mm G9
5	3 - 7	150 mm G9	150 mm G7	R+R to 150 mm G10

* R+R = Rip and re-compact.

Table B5: Possible pavement structures for the design example

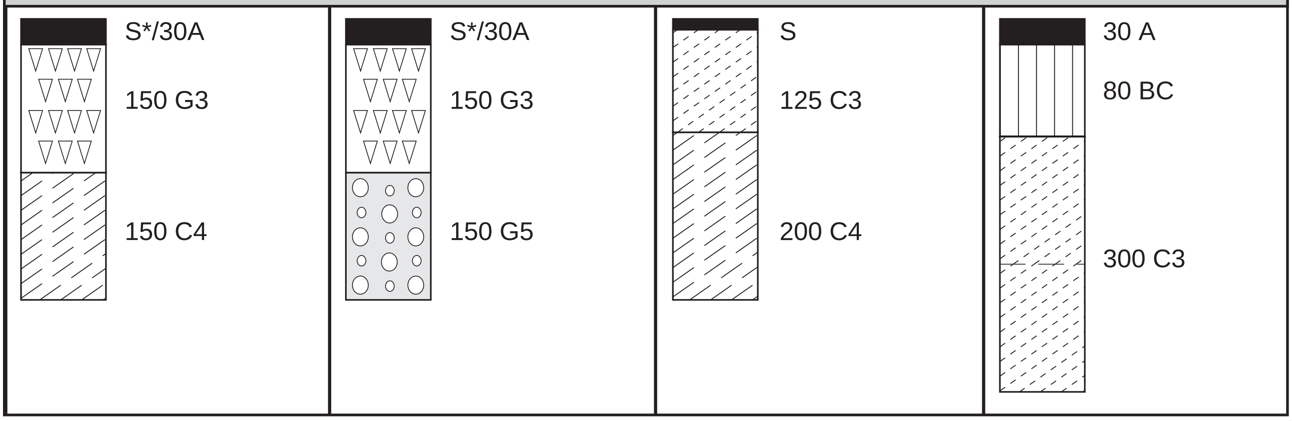


Table B6: Typical maintenance measures for the different flexible and semi-rigid pavement types over their life cycles

PAVEMENT TYPE		MAINTENANCE MEASURES	
BASE	SUBBASE	FOR SURFACING	STRUCTURAL MAINTENANCE
Granular	Granular	S1 (9 yrs) S1 (25 yrs)	40 AG (15 yrs)
	Cemented	S1 (10 yrs) S1 (25 yrs)	35 AG (15 yrs)
Cemented	Cemented	S1 (5 yrs) S1 (10 yrs) S1 (24yrs)	150 G1+S2 (15 yrs)
Hot-mix asphalt	Cemented	S1 (12yrs) S1 (25yrs)	30 AG (15 yrs)

Table B7: Present worth of costs (base year 1995)

PAVEMENT STRUCTURE	INITIAL COSTS/m² (R)	MAINTENANCE	INITIAL COSTS/m² (R)	DISCOUNTED MAINTENANCE COSTS/m² (R)			PRESENT WORTH COSTS/m² (R)		
				DISCOUNT RATE			DISCOUNT RATE		
				6%	8%	10%	6%	8%	10%
S2 150 G3 150 G5	6,00 11,00 5,88	S1 (9 yrs) 40 AG (15 yrs) S1 (25 yrs)	5,10 14,00 5,10	3,02 5,84 1,19	2,55 4,41 0,74	2,16 3,35 0,47			
	23,58			**10,05**	**7,70**	**5,98**	**33,63**	**31,28**	**29,56**
S2 150 G3 150 C4	6,00 11,70 9,26	S1 (10 yrs) 35 AG (15 yrs) S1 (25 yrs)	5,10 12,25 5,10	2,85 5,11 1,19	2,36 3,86 0,74	1,97 2,93 0,47			
	26,96			**9,15**	**6,96**	**5,37**	**36,11**	**33,92**	**32,33**
80 BC2 300 C4	25,60 18,90	S1 (12 yrs) 30 AG (25 yrs) S1 (25 yrs)	5,10 10,50 5,10	2,53 2,45 1,19	2,03 1,53 0,74	1,62 0,97 0,47			
	44,50			**6,17**	**4,30**	**3,06**	**50,67**	**48,80**	**47,56**
S2 125 C3 200C4	6,00 7,88 12,34	S1 (5 yrs) S1 (10 yrs) 150 G1 base (15 yrs) S2 (15 yrs) S1 (25 yrs)	5,10 5,10 12,90 6,00 5,10	3,81 2,85 5,38 2,50 1,26	3,47 2,36 4,07 1,89 0,80	3,17 1,97 3,09 1,44 0,52			
	26,22			**15,80**	**12,59**	**10,19**	**42,04**	**38,81**	**36,41**

B. EXAMPLE OF THE STRUCTURAL DESIGN OF A CATEGORY UD STREET

SERVICE OBJECTIVE, STREET CHARACTERISTICS AND STREET PROFILE

The residential street network needs to be designed for the same residential area as in the previous example. The layout plan and stormwater design have been completed. For the purpose of this example, the design of one of the streets will be considered. The following general information is available:

- The pavement structure for a basic access street needs to be designed.
- The route is *not* regarded as an important link in the transportation system.
- Drainage will consist of a lined channel next to the roadway
- The traffic is expected to be below 75 vehicles per day with very little heavy traffic (< 5%). Because of the absence of economic activity the growth rate will be very low (< 2%).
- The street has a steep slope in places and will be paved for erosion-protection reasons.
- Pedestrian traffic will be accommodated on the roadway and children will probably use the street as a playground.

Design pavement structures with different base types and compare them on the basis of construction costs before making a final selection.

Street category

After consideration of the characteristics listed, it is decided to design a category UD street.

Estimate design traffic

With the expected traffic (< 75 vpd), very few heavy vehicles (< 5%) and low traffic growth rate (< 2%) the calculated design traffic will not exceed 30 000 E80s even in 30 years. The street is therefore designed for an ES0,03 pavement class.

Materials

The same material availability and cost apply as for the previous example.

Structural design and pavement type selection

From the section on materials and Table 8.17 it follows that pavements with a granular, cemented, paving block or waterbound macadam base layer may be selected. The paving block structure will require a cemented subbase while the macadam base may be built with or without stabilising the subbase. The granular and cemented base layers may be used with a granular subbase. The cemented base on a granular subbase is not regarded as a sound option and will not be considered for further analysis. An emulsion-treated base layer may also be used with 1% emulsion and 1% cement (E3) at a cost of R71 (1995) per m^3. The cost for the paving block layer is R24,80 (1995) per m^2 , including the sand bedding layer.

Subgrade CBR and selected layers

The centreline CBR values are:
12,12,10,11,10,9,6,7,5,8,8,9

The subgrade is divided into two sections:
Subgrade section 1: 12,12,10,11,10,9 with a 10^{th} percentile CBR value = 9,5
Subgrade section 2: 6,7,5,8,8,9 with a 10^{th} percentile CBR value = 4,6

The design CBR for subgrade section 1 is between 7 and 15 and no selected material will be imported. The in-situ material will be ripped and compacted to a G9 standard (CBR>7) for a depth of 150 mm.

The design CBR for subgrade section 2 is between 3 and 7 and selected material will be imported. The in-situ material will be ripped and compacted to a G10 standard (CBR>3) for a depth of 150 mm and 150 mm material with a CBR>7 will be imported.

Possible pavement structures

Table B8 gives the design options for the basic access street, Category UD, pavement class ES0,03.

Cost analysis

Only the construction cost is considered in Table B9. The granular and emulsion-treated base pavements may be considered as the mechanical construction options. The cost difference of 16,6% between these two options is regarded as significant (more than 10%).

The paving block and macadam options may be considered labour-intensive construction. Although there is not a significant difference in the cost of the block paving and the 75 mm macadam options, the block paving option does offer the advantage of easy access to services if these are installed under the street, or crossing the street.

Table B8: Possible pavement structures for the basic street design example

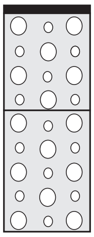
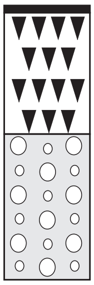
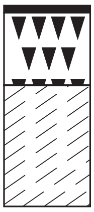
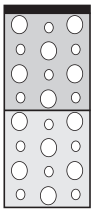
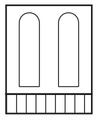
 S1 100 G4 125 G7	 S1 125 M 150 G7	 S1 75 M 125 C4	 S1 100 E3 100 G6	 60 S-B 20 SND

Table B9: Construction cost for the pavement design options for the basic access street example

PAVEMENT	CONSTRUCTION COST (R/m²)	
	LAYER COST	TOTAL PAVEMENT COST
S1 100 G4 125G7	5,10 4,26 4,10	13,49
S1 125 WM 150 G7	5,10 18,75 4,95	28,80
S1 75 WM 125 C4	5,10 11,25 7,71	24,06
S1 100 E3 100 G6	5,10 7,10 3,53	15,73
60 S-B 20 SND	22,80 2,00	24,80

APPENDIX C

MATERIAL TYPES

Materials for paved basic access streets

The specific requirements of each of the material groups for use in paved basic access streets, according to the Weinert Classification are discussed separately below. It should be noted that for arterials, traditional material specifications must be adhered to.

Basic crystalline rocks

The basic crystalline group of materials consists of those rock types which essentially have no quartz component. They include dolerite, gabbro, norite, basalt, greenschist amphibolite, etc. The pyroxene content (and possibly some of the feldspar, to a lesser extent) in basic crystalline rocks generally decomposes to form expansive, highly plastic smectite clays under wet conditions or where drainage is impeded. It should be noted that these clays may also be encountered in dry areas as a result of palaeo-climates and weathering of materials with a natural smectite component.

The presence of smectites is not always adequately shown by plasticity or strength testing and a durability mill test (Sampson and Netterberg 1988), is required to release the clays held in the aggregate. A maximum DMI of 125 is permitted for these materials. In addition a dry aggregate impact value (AIV) (Sampson and Roux 1982) should be carried out and a maximum value of 29 is acceptable. Two samples should be soaked, one in water and the other in ethylene glycol, before AIV tests are carried out on them. If the increase in AIV from the water-soaked to the glycol-soaked samples is more than five units, the material should be rejected. A first indication of the possible presence of smectite clays can be obtained by immersing fragments of the material in ethylene glycol - if they disintegrate within a couple of hours, there is a strong likelihood that smectites are present.

Acid crystalline rocks

The main rock types comprising this group in South Africa are granite, felsite, gneiss, rhyolite and syenite. Quartz makes up a significant proportion of these rocks and, as a result of its hardness and durability, it usually comprises a major proportion of the residual material. Acid crystalline materials thus generally weather to a sandy material comprising quartz, illite and kaolinite, all of these being relatively stable components. As long as the CBR strength at the expected moisture and density regimes is in excess of 50 and the plasticity index is less than 15 (Richards 1978) the material will perform satisfactorily in basic access streets. It is, however, important not to include lumps of oversize material which will break down under traffic and a maximum AIV of 30 is recommended.

A number of problems with acid crystalline rock bases have been attributed to the addition of excessive plastic fines to improve workability. The common practice of adding plastic fines to non-plastic materials for this purpose is discouraged, and where practised should be done only under strict supervision and with careful judgement.

High silica rocks

High silica rocks consist of quartzite, porphyry, hornfels and chert, where quartz or other siliceous materials - possibly with minor amounts of feldspar and clays - comprise the bulk of the rock. The materials are generally strong (provided the grading is not too uniform) and durable and if the CBR strength exceeds 50 the material should perform adequately. It is highly unlikely that durability problems will be encountered with these materials. Care should be taken with the use of chert as this is often derived from the weathering of dolomite in South Africa and is in close association with manganiferous wads and clays, which can have significant plasticities.

Arenaceous rocks

Arenaceous rocks are composed mostly of quartz, feldspars and clays, and occur as sandstone, mica schist, arkose, conglomerate and gritstone in South Africa. The clay minerals in the residual weathering products are generally inactive kaolinite and illite. Arenaceous rocks, apart from having a CBR of 50 at the expected in-situ moisture and density conditions, should have a 10% FACT value of not less than 140 kN (AIV not more than 27) and a soaked value of not less than 75% of the dry value where $N < 5$, and 55% where $N > 10$, in order to identify potentially degrading materials. A maximum fineness product of 125 with a maximum of 35% passing the 40 mesh sieve indicates an adequately durable material (Sampson 1988). If the cementing matrix is siliceous the 10% FACT limit can be dropped to 110 kN.

Argillaceous rocks

Like basic crystalline materials, argillaceous rocks are particularly troublesome as road bases, as they have a strong tendency to slake and disintegrate on exposure to moisture. The rock types making up this group in South Africa are mostly shale, mudrock, phyllite and slate and they should be used in basic access streets with care. The only argillaceous rocks which may be suitable are those relatively hard, partly weathered to unweathered, slightly baked mudrocks which could require extensive treatment (including blasting and crushing) before use and are therefore often not cost-effective for lightly trafficked roads. Certain fissile shales which can be ripped have been successfully used in lightly trafficked roads, particularly in moderate to drier areas where Weinert's N-value exceeds about 3,5. Many mudrocks are likely to slake and disintegrate in service, usually leading to distress if drainage is poor. If no other materials are available, mudrocks can be considered, but their durability according to the recommendations of Venter (1988) should be investigated. A maximum fineness product (Sampson 1988) of 125 with not more than 35% passing the 40 mesh sieve will result in an adequately durable material.

Carbonate rocks

Dolomite, limestone and marble are the main members of this group of rocks. Carbonate rocks are generally unsuitable for base courses for basic access streets, as the unweathered rocks are extremely hard and require blasting and crushing. Weathering of carbonate materials usually results in the rock proceeding directly into solution and not forming adequate residual gravel suitable for base construction. The residual material tends to be fine, consisting of insoluble residue and/or quartz or chert sand with a small quantity of hard, often coarse calcareous and cherty gravel. Where dolomite contains an appreciable quantity of chert, this forms a good gravel but it is classified as a high-silica material, as discussed previously.

Diamictites

Tillite and certain fault breccias are the sole representatives of this material group. The use of diamictite in roads has been investigated (Paige-Green 1984) and has been shown to perform well, even where existing specifications are not met. The durability of some of these materials has sometimes been a problem and the following limits for tillites are thus proposed (mostly after Paige-Green 1984):

maximum PI:	13
minimum CBR:	80
min 10% FACT:	180 kN
min soaked/dry ratio	(10% FACT): 70 if N<2; 56 if N>2
minimum Washington degradation value:	55

A maximum DMI and percentage passing the 40 mesh sieve of 125 and 35, respectively, identify materials unlikely to degrade to plastic fines in service (Sampson 1988).

Metalliferous rocks

This is a very restricted rock group in South Africa, consisting of ironstone, magnetite, haematite and magnesite, which are primarily available as the waste products of iron and magnesium mines. These rocks' very high specific gravities result in excessive haulage costs and they are probably not viable construction materials for basic access streets. If they do, however, occur close to a road project, they can be used with no problems, as they are generally unweathered, already crushed and unlikely to degrade in service. They can, however, like many waste materials, be fairly variable in quality and properties.

Pedocretes

Calcretes and ferricretes (possibly with small quantities of silcretes) are the main pedocretes used in construction in South Africa. Extensive work has been carried out on calcretes (Netterberg 1971; 1982; Lionjanga et al 1987a, 1987b) while local work on the other materials has so far been minimal. It has been shown that calcretes, in particular, exhibit self-cementation properties, and materials with high plasticities (up to 20%) and poor gradings (up to 80% finer than 0,425 mm) have performed well in roads in southern Africa.

A maximum fineness product of 480, with up to 55% passing the 40 mesh sieve is allowable for pedogenic base materials (Sampson 1988). However a minimum CBR of 50 should be obtained.

Surfacing

Bitumen surfacings are described in CSRA 1998 and range from high-quality asphalt surfacings (CSIR, NITRR 1987) to various surface treatments and slurry and sand seals (CSRA 1998). The use of asphalt surfacings (CSIR, NITRR 1987) is probably not affordable for basic access streets carrying less than 75 vehicles per day, but may be the most cost-effective surfacing in the long term, particularly when the maintenance capability is low (SABITA 1992). Traditional asphalt does not have the fatigue properties necessary to avoid cracking under the higher deflections common to light pavement structures.

It is recommended that the surfacing is applied with the prime objectives of waterproofing the underlying pavement layers and providing an all-weather, passable, dust-proof road. Aspects such as bleeding should be tolerated (Netterberg and Paige-Green 1988a) and high skid-resistance is not necessary on those access streets carrying light traffic at low speeds (a PSV of 45-50 will usually be adequate) especially in built-up areas where traffic-calming measures can be justified.

Some of the recommended limits for material properties (CSRA 1998) are probably too strict for basic access streets and relaxations may be implemented. The maximum chipping size recommended for basic access streets is 13,2mm, in order to permit comfortable use of the street for pedestrians and as a recreational area associated with high-density housing. While dusty chippings should be avoided (unless carefully precoated), the use of graded gravel or “Otta” seals (Overby 1998) should be seriously considered. The use of a graded seal results in a smaller percentage of the aggregate being wasted after crushing and a smoother, less noisy pavement. They have been found to absorb high deflections, require low maintenance, have a good resistance to hardening and be very forgiving in terms of their construction.

A relatively recent development in seals is the use of bitumen binders which have been modified by the addition of finely ground rubber or an elastomer. These add significant elasticity to the surfacing which allows it to deflect to a far greater extent without failing by fatigue. Although these binders are more expensive than traditional binders, their cost-effectiveness in terms of years of useful service is usually far greater.

The strength requirement of TRH14 (NITRR 1987) of a 10% FACT of not less than 210 kN is too strict for basic access streets, a value of 120 kN or even 80 kN being satisfactory elsewhere, provided the wet value is not less than 75% of the dry value (Netterberg and Paige-Green 1988a). It is recommended that a minimum soaked value of 80 kN should be taken as the limit for general use in basic access streets, provided the material is unlikely to disintegrate in service (e.g. mudrocks and weathered basic igneous rocks). The minimum aggregate crushing value (ACV) should not be less than 30% with a similar value for the aggregate impact value being recommended, the latter test being much simpler and quicker (Sampson and Roux, 1982). With these weaker aggregates, only pneumatic tyred rollers should be used during rolling of the surfacing stone.

Sands for slurry and sand seals should comply with the sand-equivalent requirements of TRH14 (CSIR, NITRR 1987) but the grading requirements may be relaxed slightly if no better sand is available.

It is expected that most basic access streets will be surfaced with single or Cape seals (CSRA 1998) and the following specifications are thus recommended:

Gradings:

Nominal size:	13,2 m	9,5 mm
% passing 13,2 mm	85-100	
% passing 9,5 mm	0-30	85-100
% passing 6,7 mm	0-5	0-30
% passing 4,75 mm	-	0-5

Other properties:

Flakiness index:	<30
Aggregate impact value:	<30%
Polished stone value:	<40
Dust content (passing 0,075mm):	<1,5 %

A slight relaxation of the grading and flakiness of the coarse aggregate in Cape seals is possible if no other material is economically available.

Stabilised layers

The use of stabilisation for basic access streets is probably not cost-effective much of the time, unless no other suitable materials are available. To adequately avoid carbonation of marginal materials stabilised with lime, cement, slagment or combinations of these (Netterberg and Paige-Green 1983), the stabiliser content should be equivalent to at least the initial consumption of lime (ICL) plus one percent. Bitumen emulsion stabilisation (using low residual emulsion contents) can be cost-effective for improving marginal materials.

Bituminous stabilised sand bases

Three sand-stabilisation techniques using bitumen or tar as a binder can be used (Theyse and Horak 1987) namely

- the cold wet-mix process;
- the foaming process; and
- the hot-mix process.

The sand used in the bituminous stabilised sand bases should contain between 5 and 20% material passing through the 75 mm sieve. If a particular sand type does not comply with the above criterion, filler should be added to the sand.

Cut-back bitumen of a RC-250 or RC-2 grade or a cut-back tar of 15-35C EVT grade may be used in the cold wet-mix process. For the foaming process a 60/70 penetration grade bitumen is normally used, but a 55-60 EVT tar may also be used.

The amount of binder used should preferably be determined by a factorial design method (Theyse and Horak 1987). This will ensure that a true optimum binder content is determined.

Stability criteria for basic access streets are 71 for the Hveem Rt value after moisture-vapour soak, and 200 kPa for vane shear strength (Theyse and Horak 1987). Control during construction is also done with the vane shear (Marais 1966).

Concrete blocks

Concrete or clay blocks are also viable alternatives as a surfacing/base for basic access streets. Blocks are specified as fully interlocking paving blocks, partly interlocking paving blocks and non-interlocking blocks (Clifford 1984). The crushing strength of concrete blocks should be sufficient to avoid excessive breakage during handling. It is possible to make concrete blocks for paving with local materials such as calcrete, laterite and silcrete, including the screened (and washed, if necessary) nodules (Netterberg and Paige-Green 1988a). Bedding sand should be angular with a maximum size of 9,5 mm. Material finer than 75 μm should constitute less than 15% (Shackel 1979). For higher traffic volumes, the design requirements should be considered in more detail (UTG 2 1987).

Material problems

It is important to identify problem materials timeously. Particular note should be made of potentially problematic subgrades, such as expansive clays, collapsible and dispersive soils and saline materials. Techniques have been summarised by Netterberg (1988) and in the SAICE special publication on problem soils (SAICE 1985).

The influence of these geotechnical problems associated with subgrades should not be underestimated for basic access streets where any differential movement in the soil may result in cracking. Poor or infrequent maintenance (e.g. crack-sealing, pothole-patching) will allow water access to a pavement which has not been designed to a standard which will allow for weakening by moisture ingress.

Materials for unsealed streets

The requirements of an ideal wearing course material for unsealed roads are as follows (mostly after Paige-Green and Netterberg 1987):

- stability, in terms of resistance to deformation under both wet and dry conditions (i.e. essentially resistance to rutting and shearing of the wearing course);
- ability to provide adequate protection to the subgrade by distributing the applied loads sufficiently;

- ability to provide an acceptably smooth and safe ride without excessive maintenance;
- ability to shed water without excessive scouring or erosion;
- freedom from excessive dust in dry weather;
- freedom from excessive slipperiness in wet weather without excessive tyre wear;
- resistance to the abrasion action of traffic;
- freedom from oversize stones; and
- low cost and ease of maintenance.

Unsealed basic access streets in urban areas should be designed to satisfy these requirements as far as possible, especially with respect to providing adequate protection of the subgrade. Aspects such as slipperiness, dust and oversize material should be attended to whenever possible, but the prevailing traffic and local situation should be taken into account and reductions in standards may be allowed if:

- the cost of alternative materials is prohibitive; and
- the influence of these factors is acceptable to the community in terms of frequency and consequences.

Earth streets

If the in-situ material (subgrade) conforms with the specifications of normal wearing course materials (described below), no imported layer is necessary. This is the most economic solution in many cases but it is important to ensure that the material is capable of performing adequately. The in-situ material often needs only to be shaped and compacted to a density of not less than 95% Mod AASHTO. Problems with drainage and maintenance may, however, occur if the street is not raised (at least 300 mm) above natural ground level, and it is recommended that, as a minimum, the top-soil (containing vegetable matter) be removed over a width equal to the street and side-drains, the side-drains are excavated, and the gravel from the side-drains is used to form the street. The side-drains should be below the level of the adjacent properties to avoid flooding of these properties during heavy storms.

Many proprietary products are available which claim to “stabilise” and strengthen in-situ materials, but the effectiveness and permanence of each need to be investigated before it is used. These products are mostly material-dependent, performing well with some materials and being totally ineffective with others.

Gravel wearing courses

Numerous specifications for gravel wearing course materials exist in southern Africa (Netterberg and Paige-Green 1988b), but the following performance-related specifications which were developed in southern Africa are recommended for unsealed basic access streets (mostly after CSRA 1990):

Maximum size:	37,5mm
Oversize index (I_o):	0
Shrinkage product (S_p):	100-240
Grading coefficient (G_c):	16-34
Soaked CBR:	>15 at 95% Mod AASHTOdensity
Treton impact value:	20 to 65

where

I_o = mass of material larger than 37,5mm in a bulk sample as a percentage of the total mass (Paige-Green 1988);

S_p = product of linear shrinkage and percentage passing 0,425 mm (calculated as a percentage of the material finer than 37,5 mm);

G_c = product of the percentage retained between the 26,5 mm and 2,0 mm sieves and the percentage passing the 4,75 mm sieve/100.

The plasticity-grading relationship in terms of performance is illustrated graphically in Figure C1 with the performance of the materials falling into each zone (A, B, C, etc) being identified.

All material testing follows standard TMH 1 (CSRA 1986) methods, although particle size distributions (for both G_c and S_p) are normalised for 100 per cent passing the 37,5 mm sieve (Paige-Green 1988). The performance illustrated in Figure C1 is that predicted by the two properties illustrated. Other aspects such as excessive oversize material or soft aggregate will have separate effects on the performance. An advantage of the figure is that the consequences of being slightly out from the ideal specification can be judged and the acceptably of using these materials can then be evaluated.

Specific characteristics and problems related to the various Weinert material groups identified previously are summarised below:

Basic crystalline rocks

Basic igneous rocks generally provide good wearing course materials but commonly contain rounded ("spheroidal") cobbles and boulders formed during weathering, particularly in the drier areas (Weinert N > 5). All of the cobbles and boulders must be removed (generally by hand, as they will not be broken down by a grid roller) prior to use of the material in unsealed roads, in order to eliminate their influence on riding quality and facilitate easy maintenance. In the intermediate areas (N = 2 to 5), the material often weathers to a sandy material (e.g. "sugar dolerite"), which makes a good wearing course but may corrugate quite badly. In the wetter areas (N < 2) basic crystalline rocks usually weather to a high plasticity material which becomes slippery when wet and dusty when dry.

Field evidence indicates that the use of basic crystalline rocks with few spheroidal boulders (less than 5 percent by mass larger than 37,5 mm), a plasticity index (PI) between 6 and 12 and a linear shrinkage of between 3 and 6 result in acceptable performance with minimal maintenance. A shrinkage product of less than 240 is recommended to minimise dust.

Acid crystalline rocks

These materials generally weather to a low-plasticity, sandy material with a few large, round boulders of hard rock. Corrugations are a significant problem on many streets constructed with acid crystalline rocks but regular maintenance with a simple tyre, brush or other type of drag can remove the corrugations. A PI of at least 6 will generally give good results, but in drier areas (N > 5) a PI higher than this (> 12) is recommended. Acid crystalline materials tend to be less dusty than most other materials, particularly in arid areas.

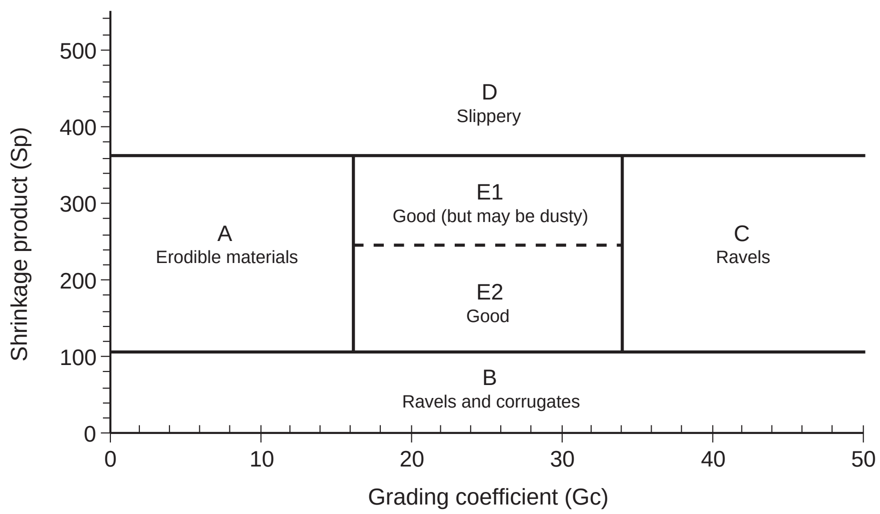


Figure C1: Relationship between shrinkage product, grading coefficient and performance (CSRA 1990)

High silica rocks

Suitable high-silica rocks for gravel wearing course are chert and quartz porphyry, both providing good wearing courses, provided the large stones are removed (especially from the chert gravels) but often producing dusty conditions - especially in wet areas. These materials often have a lack of fines and low plasticity which results in ravelling and corrugation development. Some plasticity (a linear shrinkage of more than 3) is necessary to avoid the formation of severe corrugations.

Arenaceous rocks

These materials either tend to weather to very sandy materials with little coarse aggregate or else coarse gravels with a lot of oversize, which will not break down adequately under a grid roller (particularly siliceous sandstones and quartzites). Without crushing, the latter materials are generally unsuitable. The softer sandstones (Treton value higher than 65) tend to break down and form potholes while if the linear shrinkage is less than about 3, corrugations tend to develop. The plasticity index is not a suitable indicator of performance.

Argillaceous rocks

Argillaceous rocks tend to be very stony or very clayey, depending on the weathering environment and their stage of weathering, and are invariably extremely dusty. In wet weather the argillaceous materials tend to become slippery due to their high clay-mineral content. The fissile (laminated) shales provide a relatively good wearing course, but the angular fragments frequently cause tyre damage and make the road difficult to maintain. Argillaceous rocks are therefore not recommended for wearing course unless well fragmented and treated with an appropriate dust palliative.

Carbonates

These materials seldom form an adequate quantity of material for the establishment of a viable borrow-pit in southern Africa, and are not considered as suitable wearing course materials. Many South African cherts (classified as high-silica materials) usually contain fragments of dolomite in their aggregate component.

Diamictites

Weathered diamictites have been successfully used as wearing course gravels, provided the erratic content (medium to large boulders of hard material, especially quartzite) is not too high. Their glacial origin results in a tendency to be relatively variable, and some of the tillites may form excessive plastic material that results in slippery streets when wet, while most weathered tillites give dusty conditions.

Metalliferous rocks

Because of the hardness, low plasticity and high specific gravity (resulting in high haulage costs) that are typical of these materials, they are not particularly suitable for unsealed streets. In mining areas, some waste may be suitable as a wearing course for unsealed streets but local experience will be the best guide. If the plasticity is too low (mostly the case) corrugations can be expected. Because most gravels in this class have been produced by mining and crushing, they tend to be dusty.

Pedocretes

Pedocretes are potentially the most important materials for unsealed streets in southern Africa, with calcrete being widespread in the more arid areas and ferricrete predominating in the less arid eastern areas of South Africa. Sources with good properties for unsealed road construction are, however, rapidly being used up in developing areas. The performance of calcrete wearing courses is generally good (although the materials are often extremely dusty and may contain large cobbles and boulders which are not easily broken down during construction). Ferricretes are equally satisfactory materials but like calcretes are likely to contain an excess of oversize material which should be removed or adequately broken down. Pedocretes with a PI of between 6 and 15 have provided good performance in wet and dry areas of southern Africa.

Stabilised earth streets

The use of lime- or cement-stabilised earth streets is common in certain parts of Europe (Kézdi 1979) and could possibly be considered for local application. No research has as yet been carried out on these materials locally, but the problem of carbonation (Netterberg and Paige-Green 1983) and the potential loss of strength resulting from this process may be significant. However, it is considered that, even with carbonation, the ravelling and abrasion will probably be less than with a natural gravel although maintenance may be a problem as the loose material is likely to be non-plastic (and very dusty). Further research into this possible solution is necessary but, for the present, its use should be based on sound engineering judgement.

Dust palliatives

It is often difficult to obtain materials which will provide a dust-free surface. Dust palliatives are chemical or bituminous agents which are mixed into the upper parts or sprayed on the surface of a gravel wearing course and bind the finer portions of the gravel, thus reducing dust. A wide variety of dust palliatives is available and economic analyses need to be carried out for each product in each situation to determine their viability. The social impact of dust is, however, difficult to quantify in economic terms but should not be neglected from any analyses. Recent research has shown that the deliquescent products (usually calcium chloride), the ligno sulphonates, certain polymers and some liquid chemical stabilisers (LCS) can provide good dust palliation, cost-effectively. Dust palliatives have been fully discussed earlier in this chapter.

Most dust palliatives have so far been tested on rural and inter-urban roads. The environment in a residential area is completely different in terms of drainage, traffic volumes and speed, intersections, etc. One aspect which needs to be considered is the ease of maintenance of treated roads. Road surfaces which are treated with products that do not penetrate into the layer cannot easily be maintained once potholes and ravelling initiates. Similarly, materials which strengthen the road considerably (e.g. liquid chemical stabilisers) do not allow grader maintenance should large stones protrude from the surface, as they are not easily plucked out and lead to unacceptable roughness. The optimum solution is to mix appropriate products through the layer, ensuring that all large stones (greater than 37,5 mm) are removed from the wearing course material.

No general guideline for dust palliatives is currently available but a research project involving a performance-related study of a limited number of generic dust palliatives is in progress.

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Chapter 9

Water supply

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SCOPE

These guidelines cover aspects that need to be considered when planning and implementing water supply projects for existing residential areas and developing communities. The guidelines will also be of assistance where a Water Services Authority compiles a Water Services Development Plan (the latter forms part of a municipality's Integrated Development Plan).

The guidelines assist in determining and setting objectives, developing a strategy and identifying the required planning activities for implementing water services. Technical guidelines are given for use in feasibility studies and the detailed design of water supply elements.

The guidelines form part of a planned series of management guidelines intended for use by decision-makers. The series of guidelines is shown in Table 9.1.

INTRODUCTION

Water services (i.e. water supply and sanitation) in South Africa are controlled by the Water Services Act (Act 108 of 1997) and the National Water Act (Act 36 of 1998). The Water Services Act deals with water services provision to consumers, while the National Water Act deals with water in its natural state.

Central to the supply of water to a community is the Water Services Development Plan of the relevant Water Services Authority, which is required in terms of the Water Services Act. The Water Services Development Plan defines the minimum as well as the desired level of water service for communities, which must be adhered to by a Water Services Provider in its area of jurisdiction. It describes the arrangements for water service provision in an area, both present and future. Water services are also to be provided in accordance with by-laws made in terms of the Water Services Act.

Engineers and other decision-makers within a Water Services Authority, and those working for and on behalf of the Water Services Authority, should be aware of the social and organisational constraints in the provision of potable water. The issues relating to these constraints must be addressed in the objectives of any water supply project, keeping in mind that the sanitation arrangements for a community are inextricably bound to the process (see Chapter 10).

The principles of sustainability, affordability, effectiveness, efficiency and appropriateness should be kept uppermost in supplying water to a community. These and other important issues are dealt with under the relevant headings in this chapter.

Table 9.1: Management guidelines for water service providers (Palmer Development Group 1994)

NAME	TYPE	AUTHOR
URBAN SERVICE PROVIDERS		
Organisational arrangements for service providers	Institutional	PDG
Consumer profile and demand for services	Economic	PDG
Preparation of a water services development plan	Planning	PDG
Water supply tariff setting	Finance	PDG
Sanitation tariff setting	Finance	PDG
Reporting	Management	PDG
Guidelines for private sector participation	Institutional	Pybus
RURAL SERVICE PROVIDERS		
Establishing effective service providers	Institutional	
IMPLEMENTING AGENTS		
Guidelines for local authorities and developers (urban)	Planning	PDG
Series of guidelines for rural areas	Technical	CSIR
DESIGNERS		
Red Book	Technical	CSIR
COMMUNITY LEADERS		
Guidelines for community leaders (urban)	General	PDG
Guidelines for community leaders (rural)	General	CSIR

THE IMPORTANCE OF HYGIENE PROMOTION IN WATER SUPPLY AND SANITATION

Introduction

The principal purpose of programmes to improve water supply and sanitation is to improve health. On the other hand, the mere provision of water and sanitation infrastructure will not, in itself, improve health. To get the maximum benefit out of an improved water supply and sanitation infrastructure, people need to be supported with information that will enhance these benefits. This form of information, and the imparting of skills, is called hygiene education. Hygiene promotion and education provides people with information that they can use to change their behavioural patterns in order to improve their health. Changes in behaviour do not come automatically, but also have a motivational component. In many instances incentives are necessary to induce a change in behaviour, the major incentive being the benefit derived from changed behaviour.

An important lesson learnt during the International Drinking Water Supply and Sanitation Decade is that good coverage – providing a large number of people with access to facilities – does not equal success or sustainability. Because water supply and sanitation facilities are subject to misuse, non-use, or breakdown, international donors and national governments alike have come to recognise that the sustainability of systems is of critical importance. Apart from a sense of ownership of the facilities, it also means that communities should adopt hygiene practices that will help them realise the health benefits of water supply and sanitation improvements. Hygiene promotion and education is a key component of the effort to achieve these health benefits.

To achieve sustainable water supply and sanitation development requires effective complementary inputs such as community participation, community capacity-building and community training. International trends and research have indicated that hygiene promotion and education plays a major role in breaking down the transmission of diseases that are affecting many rural communities in the developing world.

In South Africa it is essential to understand the attitudes and behaviours of developing communities towards water, sanitation and hygiene. Most developing communities rely on the government to make sure that their water supply and sanitation projects are sustainable, but it is necessary for the community itself to contribute to the sustainability of its projects, as well as to the development of an appropriate hygiene-promotion and education programme. It is at community level that real decisions on hygiene promotion and education should be made, but these communities need information to be able to make decisions reflecting their aspirations, desires and needs.

For guidelines on implementing a project with the above elements, see Appendix A of Chapter 10.

What is hygiene promotion and education?

Hygiene promotion and education is not about coercion, but about bringing change in the behaviour patterns of people, to make them aware of the diseases related to unhygienic practices, poor water supply and improper sanitation. It forms an integral part of any water and sanitation development programme.

Hygiene promotion and education comprises a broad range of activities aimed at changing attitudes and behaviours, to break the chain of disease transmission associated with inadequate water supply and sanitation. It is the process of imparting knowledge regarding the links between health, water and sanitation, and seeking to provide people with information that they can use to change their behavioural patterns so that they can improve their health. It is about keeping well, about a better quality of life and about recognising that the majority of illnesses that kill children can be associated with poor sanitation practices and inadequate or unsafe water supplies. It is a primary intervention that, like immunisation but much more cheaply, aims at preventing illness or minimising the risk of infection.

A definition of hygiene promotion and education that emphasises activities aimed at changing attitudes and behaviours must recognise that behavioural change cannot be effected from outside the communities. The individuals in the community must want to change, and only they can effect sustainable change. The role of the external agent can be only that of a catalyst and of providing (or broadening) awareness. Furthermore, the role of women cannot be overemphasised. Women are the latent forces for change in local communities, and their empowerment and involvement are prerequisites to the success of a community-based health or hygiene education and awareness programme or campaign.

It is now recognised worldwide that hygiene promotion and education is an important channel to link newly installed facilities to improved health. Improved water supply and sanitation systems will reduce the persistence and prevalence of diseases.

PLANNING: OVERVIEW

General

The provision of water to a community has to follow the same route as any other project, in that it has to go through a series of distinct stages between the initial conceptualisation and the time when the project is completed. These stages, shown in Figure 9.1, can be summarised as follows:

- *Identification and preparation* comprise the pre-investment planning stages.
- *Approval* is the stage at which decision-makers, including financiers, determine whether or not a project will become a reality.
- *Implementation* is the stage at which detailed designs are completed and the project facilities are built and commissioned; supporting activities such as staff training are also undertaken.
- *Operation* is the stage during which the project facilities are integrated with the existing system to provide improved services.
- *Evaluation*, the final stage, determines what lessons have been learned so that future projects can be improved accordingly.

It is important that the project be undertaken within a framework of clear objectives, aimed at ensuring maximum operational effectiveness, as well as sustainability on completion.

Technical guidelines should be assessed in the context of the operational goals set for the water supply, and adjustments made to take into account factors such as levels of income, availability of funds and the ability of the community to operate and maintain the service. Sustainability of the service is the most important criterion that must be addressed in the planning phase.

The planning process should produce reports that define the purpose and objectives of the water supply and set the broad strategy for reaching the objectives. These reports act as overall guidelines and assist in the generation and selection of the alternative technologies that could be used in the provision of the water supply. The community to be served should be involved in the planning process.

Where the upgrading or rehabilitation of an existing water supply scheme is contemplated, a thorough investigation of existing supply arrangements is required. Eliminating water theft, reducing unaccounted-for water and improving recovery mechanisms could render capital works unnecessary, or postpone them.

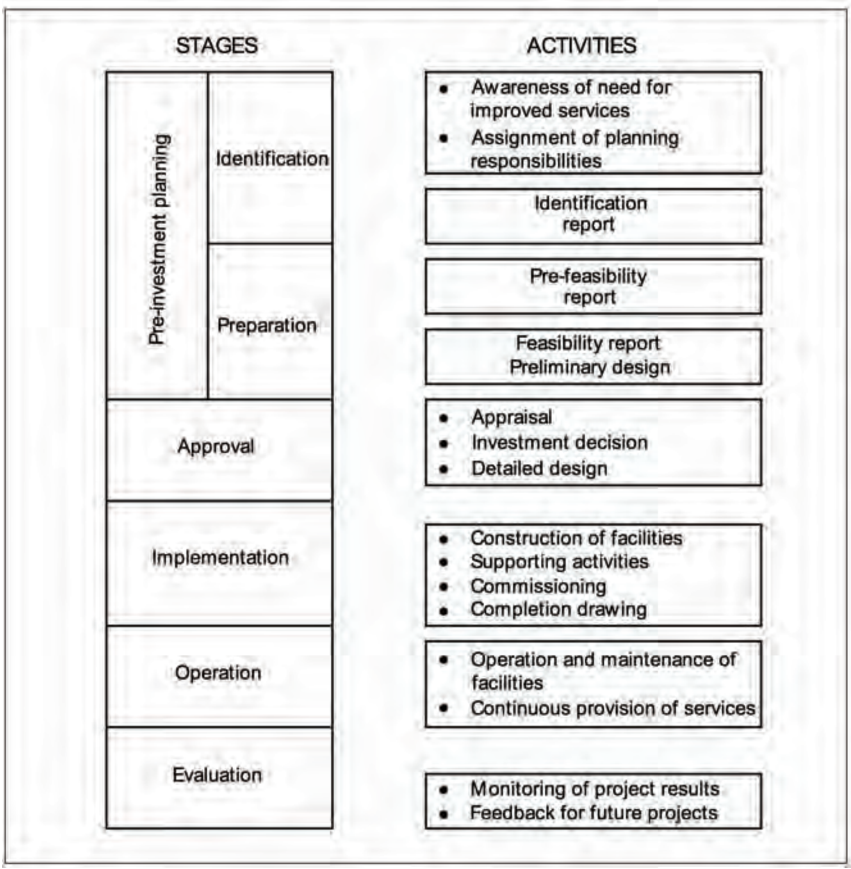


Figure 9.1: Development stages for water supply and sanitation projects

Purpose of the water supply

Establish the purpose of the water supply. Why is the water supply needed? Who will use the water and for what activities? What is the problem with the current situation and how will the proposed water supply project alleviate the problem?

Objectives

Set broad objectives, or goals, first for the operational phase and then for the project phase. It is important to look at operational objectives first, and use these to establish the objectives for the project phase, otherwise there is a risk that the water supply system will operate inefficiently, even if the project phase was completed successfully.

The objectives of a water supply project should include the following:

- the provision of water for domestic consumption and personal hygiene in terms of the Water Services Authority's by-laws (government policy requires that a minimum of 25 litres per person per day be provided);
- the improvement of the quality of the existing supplies (protection of the sources being the first consideration);
- the improvement of the availability of water to the community (both reliability and accessibility);
- community involvement (acceptability) and commitment;
- the improvement of public health;
- the improvement of the living standards of the community;
- the development of local technical, financial and administrative skills; and
- the improvement of the economic potential of the community (e.g. small-scale agriculture and industries).

Strategies

An overall strategy is needed to guide the project through various stages into the operational phase.

By-laws

Note should be taken of the by-laws of the Water Services Authority. The following aspects are of particular importance where Water Services Development Plans are incomplete or unclear:

Administration

The community should be involved in the planning, implementation and maintenance phases of the project (preferably through an independent committee of community representatives).

Finance

Subsidisation of the scheme by bodies outside the community is restricted to the provision of the basic level of service prescribed in government policy documents. The community must also be able to bear the operational costs involved. There are, however, exceptions to the rule, which can be found in the policy documents.

No water supply system should be planned in the absence of a tariff structure and expense-recovery mechanism, agreed to by the client community. The client community must be able to pay for its basic operation and maintenance, with due regard to the free basic water policy of the National Government.

Development impact

Maximum use should be made of local manpower and materials, with training given where appropriate. Where possible, local contractors and entrepreneurs should be employed. However, the technologies employed – including labour-based construction methods – should be cost-effective.

Health

The improvement of the quality of services should be driven by increased community awareness of health-related problems and their causes. For example, improvements in living standards and public health in a community may be impossible to achieve unless hygiene education is provided and sanitation improvements are made concurrently with an improvement in water supply.

Planning activities

The objectives, strategy and policies must provide sound guidelines for formulating and executing the activities, tasks and sub-tasks required to reach the given set of objectives.

The completion of an activity should result in an objective being met. For example, an objective could be the commissioning of a single element of the water supply that is needed to achieve the overall purpose of the whole scheme.

PLANNING: REPORTS

Project reports

In the absence of other guidelines on a project report, the format and contents of the reports should follow the following format.

Feasibility reports

Feasibility reports should cover any factors that could be relevant to the detailed planning and design of a new water supply scheme, or the upgrading of an existing one. Some analyses that should be considered in the feasibility study are given in the documents referred to in Table 9.1.

Water demand

Future water demand is one of the key issues in water supply planning. The following important points regarding the demographic and economic situations determining future water demand should correspond with the contents of the Water Services Development Plan.

The demographic and service information required includes:

- the current population;
- the number of households;
- the number of residential consumer units;
- the incomes related to these consumer units;
- the number and type of non-residential consumer units;
- current levels of water service;
- current consumption; and
- the demand for services, in terms of willingness to pay for the services desired.

The information required to make proper projections of future requirements includes:

- population growth;
- economic growth;
- growth in number of consumer units;
- level of service provided to residential consumer units;
- changes in income levels of residential consumer units;
- changes in consumption per consumer unit;
- effects of water-metering programmes; and
- weather patterns and climate.

Water conservation and demand management (DWAF 2002)

One of major impediments to the implementation of water conservation and demand management at a local level is the lack of social awareness and understanding about these topics among both consumers and water service institutions/authorities. If not implemented in an integrated, targeted and strategic manner, social awareness campaigns will have limited success in achieving the desired behaviour change in water use patterns. Social awareness campaigns need time, energy and resources, and those promoting water demand awareness need to adopt a single, consistent message.

Attempts to implement water conservation and demand management have generally focused on narrow, technical solutions. However, successful implementation is as much about raising awareness as it is about technical interventions. As social awareness is often implemented in conjunction with other measures, gauging its impact on consumer demand is not easy, making it a less attractive option compared to those that provide quick, clear and good results. Awareness-raising is also perceived either as difficult to implement, or simply about making posters or pamphlets. Those involved in raising awareness about water issues do not approach it from the type of marketing perspective needed to sell a product or a concept.

Raising awareness about water demand has to be approached in a strategic manner. Changing the mindsets and behaviour of both water users and managers is a fundamental component of water conservation and water demand management. It is one of the first steps that must be taken in an integrated water demand management strategy in order to achieve the acceptability and buy-in necessary for more technical measures to succeed. Water users and consumers should not be the only targets of education and awareness campaigns; rather, campaigns should be specifically targeted at all stakeholders, including water services institutions, local government, etc.

Raising awareness about water conservation and water demand management issues facilitates changes in behaviour, as knowledge about the subject increases through the education of stakeholders. The effectiveness of any awareness campaign is ultimately measured by the results of the implemented water conservation and water demand measures.

To be successful, any awareness/education campaign has to be *integrated*, *ongoing*, *relevant* and *targeted*. Preliminary research is therefore necessary to develop an understanding of the characteristics, conditions and dynamics of the context/community in which awareness raising needs to be conducted. The Knowledge, Attitudes and Practices (KAP) survey tool provides a model for facilitating change on an individual basis, to incorporate new practices that are being introduced.

General

Several methods may be used to predict the future population. It is important to note that conditions in this country differ to a great extent from those in other countries, and population growth is influenced by a host of demographic factors, which include migration and urbanisation. It is therefore considered important to consult demographers and town planners, as they are best equipped to deal with the issues of socio-economic planning and hence the future population of a given area. Designers should take note of the consequences of accepting an excessive growth rate, but cognisance should also be taken of the characteristics of the study area. Factors like employment opportunities, available residential area, infrastructural services and HIV/Aids can have a significant effect on growth rates.

Water demand figures adopted for design purposes should be based on a projected value for, say, 20 years hence.

Design periods for the integral components (i.e. purification works, reservoirs, pumps and mechanical components, electrical components and main pipelines – outside reticulation) should not exceed 10 years.

Design water demand values for communities are given under the section "Design Criteria for Water Distribution and Storage Systems" in this chapter. These are average daily figures and the design demand should be based on the peak daily demand at the end of the economic design life of the project (i.e. the point where more capital will be required to expand the facilities).

Designers should note that, in adopting water demand figures for a specific design, cognisance should be taken of local factors such as income level, climate and water charges, when interpolating between the upper and lower limits given in this chapter.

Wastewater disposal

It should be borne in mind that increasing the quantity of water supplied to an area also increases the quantity of wastewater for disposal. It is therefore imperative, in the planning stage, that present wastewater disposal practices be evaluated to assess whether these methods can cope with an increased load arising from increased water usage. This aspect is dealt with in Chapter 10 (Sanitation). The recycling of wastewater can reduce the demand on water significantly and could be implemented where feasible and where human health will not be compromised.

A broad approach to the contents of project reports is given in Table 9.2.

Project business plans

The business plan of a project describes a strategic programme-based approach to water service delivery by:

- giving details of the project;
- demonstrating how it will conform to national policy;
- describing how it will be implemented and managed;
- showing how progress will be measured against goals specified; and
- discussing a funding strategy and sustainability.

Business plans normally have the following components:

- an introduction, describing the purpose of the business plan;
- a description of the management structure;
- a project description, which should include the technical details;
- the details of conformity with national policy and other guidelines of funders;
- a table of cost estimates; and
- time plans.

The format and content of business plans is usually prescribed by funding agencies and government departments.

WATER QUALITY

General

Water quality refers to the presence of living organisms or substances suspended or dissolved in water. Water used for domestic purposes needs to be of an acceptable quality and should have a certain amount of dissolved salts present, both for taste and

Table 9.2: Broad approach to contents of planning reports

TOPIC	PRESENT SITUATION
Climate	Rainfall, evaporation, seasonal changes.
Hydrology	Sources of uncontaminated water. Depth of water table. Recharging of groundwater. Presence of streams, rivers, dams.
Geology	Prevalent rock and soil types. Likely existence of impervious and pervious layers. Geological faults. Potential for boreholes. Dam sites.
Population	Existing distribution and numbers. Horizontal and vertical distances from water sources. Migrant labour practices. Age distribution.
Prevalent diseases	Water-related (typhoid, cholera, gastro-enteritis, scabies, bilharzia, malaria). Nutrition-related (malnutrition, kwashiorkor).
Financial resources	Average expenditure per household. Savings.
Institutional structures	Leadership (chief, councillors, government officials). Committees and clubs; household heads. Fieldworkers (agricultural, health). Responsibility for water supply, sanitation, health. Role of women and youth.
Existing water supply and distribution	Springs, pumps, water vendors, tanks, boreholes. Reliability and demand. Costs. Type and age of piping. Present consumption.
TOPIC	**FUTURE DEVELOPMENTS**
Population	Expected rate of increase and controlling factors. Informal settlements (urbanisation). Age distribution. Migrant labour practices.
Public facilities	Schools; clinics, hospitals; transport; recreation centres.
Commercial and Industrial	Factories, shops, offices, restaurants.
Financial prospects	Improved per capita income. Savings.
TOPIC	**COMMUNITY VALUES, ATTITUDES, NEEDS AND SKILLS IN RESPECT OF:**
Water supply	Quality; distance; quantity (expected consumption after upgrading).
Sanitation facilities	Toilets, disposal systems; washing hands; disease transmission.
Expertise and skills	Administrative; technical; financial; leadership.
Need priorities	Domestic water; sanitation; public facilities; commercial enterprises; industrialisation.
Education and training needs	Health; water supply schemes; maintenance.

to minimise the corrosive potential of the water. Furthermore, it has been estimated that only one out of every 20 000 strains of bacteria is pathogenic, and the mere presence of bacteria in drinking water is not necessarily a cause for concern. The approach to water quality control in water supply projects should therefore include the following steps:

- the protection of all components (including the source, storage units and pipelines) against possible contamination by pathogenic organisms;
- the improvement of the existing water quality to ensure aesthetic acceptability (removal of turbidity and unpleasant taste); and
- the education of consumers regarding basic precautions for the collection, storage and use of water.

Aspects of water quality that have a bearing on these requirements are discussed in the following sections.

Diseases associated with water

Water development projects are intended to improve the quality of the human environment. However, unless well planned, designed and implemented, a water project may bring about a decrease in one type of disease but cause an increase in a more severe type. This may be especially true of projects designed to improve local agriculture.

Hence, one of the chief concerns of water quality control is the spread of diseases where water acts as a vehicle. The World Health Organisation (WHO) estimates that 80% of illnesses in developing countries are related to waterborne diseases.

Information on water related diseases is available in various textbooks.

In order to minimise water related diseases the following should be observed:

- the disinfection of domestic water supplies;
- the provision of well-designed and -constructed toilets;
- an increased quantity of water for domestic use;
- the provision of laundry facilities, thereby reducing contact with open water bodies; and
- the provision of adequate drainage and the disposal of wastewater.

Water quality

Water for human consumption must comply with the requirements of SABS 241. This standard provides for a Class 0 or Class 1 water – the two classes intended for lifetime consumption – and a Class 2 water, intended for short-term consumption, in relation to the physical, organic and chemical requirements as specified. All classes of water must comply with the specified microbiological requirements.

Stability of water supplies

Water that is put into distribution pipelines should be neither corrosive nor scale-forming in nature. Corrosive water may lead to corrosion of the pipelines, fittings and storage tanks, resulting in costly maintenance, and/or the presence of anti-corrosion products in the final water being delivered to the consumer.

WATER SOURCES

When planning a water supply scheme for an area, the potential sources of water should first be assessed. Consideration should be given to the quantity of water available to meet present and future needs in the supply area, as well as to the quality of the water. Water that is unfit for human consumption will need to be treated before being distributed.

Water for human settlements can be obtained from one or more of the following sources:

- springs;
- wells and boreholes;
- rainwater;
- surface water – rivers and dams;
- bulk-supply pipelines; and
- a combination of the above.

Springs

A spring is a visible outlet from a natural underground water system. Management and protection of the whole system, including the unseen underground part, is essential if the spring is to be used for water supply. The seepage area can be identified by visual inspection of the topography, and the identification of plant species associated with saturated ground conditions. The area can be fenced off, surrounded by a hedge, or just left under natural bush and marsh vegetation. Gardens and trees can be safely planted some distance downstream of the spring, but not within the seepage area above the eye of the spring. The conservation of wetlands or spring seepage areas is an extremely important and integral part of spring water development and management.

Generally, springs fall into three broad categories. These are:

- *Open springs*: occurring as pools in open country. Some form of sump or central collection point from which an outlet pipe can be led is all that is required. It may sometimes be necessary to protect the eye of the spring.

- *Closed springs*: the more common form of spring found in rolling or steep topography. In this case a "spring chamber" is constructed around the eye of the spring, completely enclosing it. Some form of manhole should be provided so that desilting, routine maintenance, and inspection of the pipe intake can be undertaken. It should not be the function of the spring chamber (cut-off wall, spring box or V-box) to store water, since a rise in the chamber's water level above the eye of the spring can result in the underground flow of water finding additional outlets or eyes.

  The spring chamber in Figure 9.2 should be designed according to the principles of underground filters. Provide a graded filter or filter cloth between the in-situ material and the outlet pipe.

- *Seepage field*: where the spring has several eyes or seeps out over a large area. In this case, infiltration trenches are dug and subsoil drains constructed. The drains feed the spring water to a central collector pipe. Subsoil drains can be made of stone, gravel, brushwood, tiles, river sand, slotted pipes, filter material or a combination of the above.

The outlet pipe from a protected spring is usually fed to a storage tank, which keeps the water available for use. The storage tank should have an overflow pipe that is below the level of the spring outlet in the case of gravity feed.

The area immediately above and around the spring outlet or protection works (see Figure 9.3) should be fenced, to prevent faecal contamination by humans and animals. A furrow and berm should be dug on the upstream side of the outlet, to prevent the direct ingress of surface water into the spring after rains.

The reliable yield from a spring is estimated by measuring the outlet flow rate during the driest months of the year (August/September in summer rainfall areas, February/March in winter rainfall areas). The reliable yield is then calculated by multiplying this flow rate by a factor. This factor (see Table 9.3)

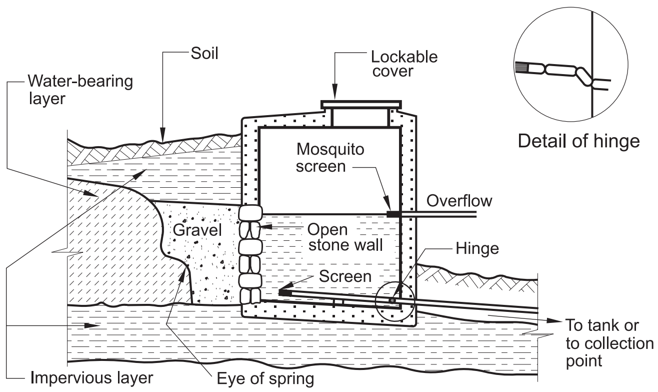


Figure 9.2: Details of spring chamber (Cairncross and Feachem 1978)

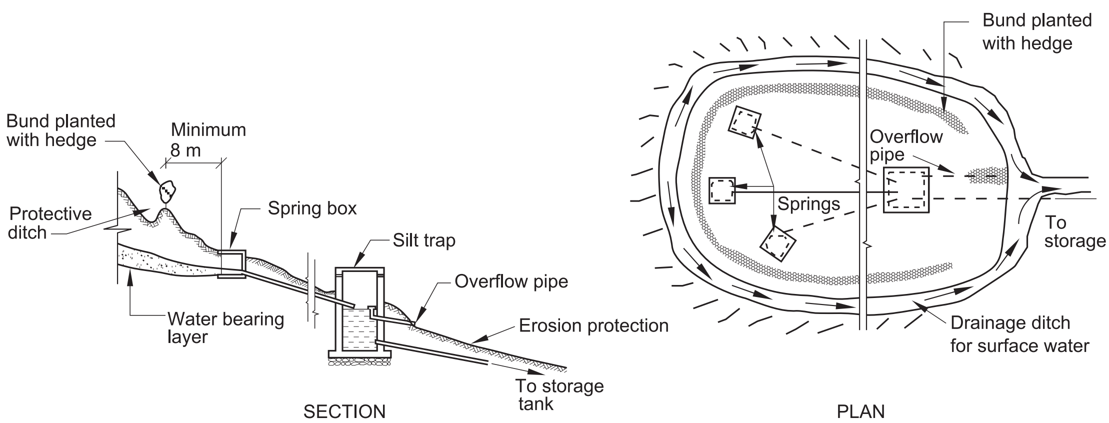


Figure 9.3: Layout of spring protection works for multiple spring eyes (Cairnross and Feachem 1978)

Table 9.3: Factors for obtaining reliable yield estimates of spring water

RAINFALL DURING PREVIOUS WET SEASON	FACTOR
Above average, extending into normally dry season	0,25
Above average	0,35
Average	0,50
Below average	0,65
Below average, longer than usual dry period	0,80

depends on a number of variables, including geology, soil types, land use, and hydrological characteristics. As a first approximation the following factors may be used, but it is advisable to try to obtain additional information where possible.

Usually, the local populace can provide information on whether the spring ever dries up, or how many containers can be filled in an hour for the worst drought years.

It is also reasonable to assume some level of risk, especially since during at least 90% of the year better flow conditions than the reliable yield can be expected.

Wells

Where the underground water does not emerge above the natural surface of the ground, this water can be accessed by digging a well in the case of shallow depths, or drilling a borehole when the water level is deep (i.e. greater than 15 m).

Hand-dug wells

A well is a shaft that is excavated vertically to a suitable depth below the free-standing surface of the underground water. It is usually dug with hand tools, and consists of a well head (the part visible above ground), a shaft section and the intake (the area where water infiltrates).

The well head's construction will depend on local conditions but must be built in a way that contributes to hygiene and cleanliness. The well lining should extend above the ground surface, to prevent contaminated surface water from running down into the well. For this reason, and to prevent subsidence, the space between the lining and the side of the shaft should be backfilled and compacted. A concrete apron, sloping away from the well, should preferably be cast around the well.

It is necessary to provide some form of lining to prevent the walls of the shaft collapsing, both during and after construction. Types of linings used include:

- reinforced concrete rings (caissons);
- curved concrete blocks;
- masonry (bricks, blocks or stone);
- cast-in-situ ferrocement;
- curved galvanised iron sections; and
- wicker work (saplings, reeds, bamboo, etc).

The well must be sunk sufficiently deep below the free-standing surface of the groundwater to form a sump in order to provide adequate water storage, to increase the infiltration capacity into the well, and to accommodate seasonal fluctuations in the depth of the water table. The larger the diameter of the hole, the faster it will recharge, depending on the characteristics of the aquifer. Joints between the linings can be sealed with mortar or bitumen above the water table, but left open below it.

The intake section is that part of the shaft in contact with the aquifer. Joints in this section must be left open. It is advisable to cover the bottom of the well with a gravel or stone layer to prevent silt from being stirred up as the water percolates upwards, or as the water is disturbed by the bucket or pump used for abstraction.

The well should be covered with a slab and equipped with a suitable pump or bucket and a lifting mechanism.

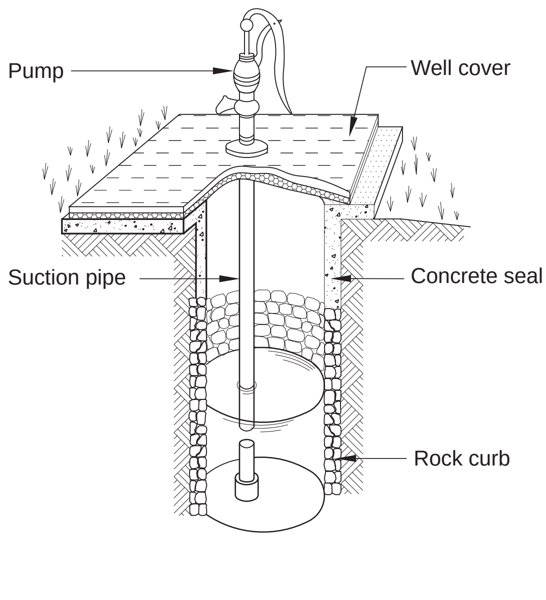


Figure 9.4: Hand-dug well (IRC 1980 & 1983)

Tube wells (also called bored wells)

In sandy soils, the hand-digging of wells is problematic and expensive since loose sands tend to collapse. Therefore hand-digging in sandy soils is not recommended as cheaper, more efficient methods are available. These methods include jetting, hand-drilling and augering of small-diameter holes (50 to 500 mm). The holes are lined using uPVC or mild steel casings to prevent collapse. The section below the water table is fitted with some form of well screen to allow for filtration of the groundwater while preventing the ingress of silt.

As with hand-dug wells, the tube well should be covered with a slab and equipped with a suitable pump and concrete apron. Specially designed buckets that can fit into the tubes and be winched down to the water level are still commonly used in tube wells. Certain designs of bucket eliminate the need for handling and, hence, the possibility of polluting the well water with germs, etc, from unwashed hands.

Boreholes

Generally, underground water is of a better quality, in terms of bacteria and suspended solids, than surface sources, and its supply is often more reliable. For these reasons human settlements throughout history have shown a preference for underground water, when available, for domestic water supplies. In all cases, groundwater should be analysed to determine its fitness for human consumption as well as its possible effect on pipe systems.

When the water table occurs at a great depth and/or in rock formations that do not facilitate the construction of hand-dug wells, a relatively small hole can be drilled using mechanical equipment. With the proper equipment, such boreholes can be sunk to depths of 100 m or more, if required.

The borehole should be drilled by a reputable drilling contractor registered with the Borehole Water Association of South Africa. The drilling should also be executed in terms of accepted procedures and standards, e.g. the Association's publication *Minimum Code of Practice for Borehole Construction and Pump Installation.*

The diameter of the hole should suit the size of the casing to be installed, plus any temporary casing required to keep the hole open during drilling and gravel-packing. For most hand-pump installations a casing diameter of 100 to 110 mm is adequate, while submersible pumps normally require a minimum diameter of 120 mm, and preferably at least 150 mm.

As with hand-dug wells and tube wells, it is important to prevent surface water entering the borehole, and to drain any excess water from the borehole site. If necessary, a concrete apron or collar should be provided. The installation of a sanitary seal provides effective protection against aquifer pollution via the borehole annulus. Wherever possible, a local resident should be trained to maintain the borehole and borehole pump and to alert the appropriate authorities when major breakdowns occur. Water level measurements should be taken regularly and recorded, to ensure the pump is submerged at all times and provide early warning of source depletion.

Siting of wells and boreholes

The presence, amount and depth of deep underground water cannot normally be predicted beforehand with a high degree of accuracy. Boreholes and wells previously sunk in the area could give valuable information as to the depth and amount of water available. Trained geoscientists (e.g. hydrogeologists or geophysicists) are able to establish the most favourable sites by using techniques such as aerial photograph interpretation and geophysical exploration – e.g. electrical resistivity, magnetic, seismic and gravimetric measurements. National and regional groundwater maps providing synoptic and visual information on South Africa's groundwater resources are available from the Water Research Commission and the Department of Water Affairs & Forestry. These maps are not site-specific and cannot be used for borehole siting or any site-specific groundwater conditions, but are an aid in determining borehole prospects and other groundwater related information such as quality.

Groundwater is vulnerable to pollution. All boreholes not correctly equipped should be properly closed. As a minimum guideline, boreholes for domestic use should be at least 30-50 m away from potential pollution sources such as on-site toilets, cattle kraals or cemeteries; however, this general rule must be considered against site-specific conditions and circumstances.

Determination of yield

Once a successful borehole has been established, it is important to carry out tests to estimate the yield likely from that borehole. The type of test and its duration must be chosen to suit the level of reliability required. Recommendations in this regard are given in Table 9.4. These recommendations represent the minimum requirements, and can be altered to suit the situation (extract from *Test pumping standards for South Africa* published by the Ground Water Division of the Geological Society of South Africa).

All the aspects addressed above are covered in a document *Minimum standards and guidelines for groundwater resource development for the community water supply and sanitation programme* published by the Department of

Table 9.4: Recommended test and duration to estimate subsurface water yield

USE OF WATER	TEST	DURATION	RECOVERY TEST
Stock or domestic	Extended step	Total 6 hours	Up to 3 hours
Hand pump	Extended step	Total 6 hours	Up to 3 hours
Town water supply Low-yield borehole	Step Constant discharge	4 x 1 hour 24 hours	- Complete
Town water supply High-yield or main borehole	Step Constant discharge	4 x 1 hour 72 hours or more	- Complete

Water Affairs & Forestry (1997). More recently, the South African Bureau of Standards embarked on the development of a set of South African Standard Codes of Practice (SABS 0299 series) for the development, maintenance and management of groundwater resources.

Rainwater

Rainwater can be collected and stored. The harvesting of rainwater from roof runoff can supplement domestic supplies, even in semi-arid areas. In particular, rainwater can be harvested not only for domestic use, but also to provide water at remote public institutions like schools and clinics, as well as resorts. Usually the limit is not the amount of rainfall that can be collected, but the size of the storage tank that will provide a sustained supply during periods of little or no rainfall. It should, however, be considered a supplementary supply for non-potable use since it could pose a health risk.

Rainwater collection from roofs constructed from corrugated iron, asbestos sheeting or tiles is simple. Guttering is available in asbestos cement, galvanised iron, uPVC, plastic or aluminium. The guttering and downpipes can be attached directly to the ends of rafters or trusses, and to fascia boards.

Because the first water to run off a roof can contain a significant amount of debris and dirt that has accumulated on the roof or in the gutter, some mechanism (such as that in Figure 9.5) to discard the first flush is desirable. In addition, the inlet to the storage tank should be protected with a gauze screen to keep out debris, as well as mosquitoes and other insects or rodents.

Materials commonly used for rainwater tanks include corrugated iron, glass fibre, asbestos cement, high-density polyethylene (all prefabricated types) or ferrocement, concrete blocks, masonry, reinforced concrete, and precast concrete rings (tank constructed in-situ). Subject to the availability of a suitable mould, ferrocement construction is one of the most economical options at present. Ferrocement construction without the use of a mould is also possible, however.

Larger quantities of rainwater may be collected from specially prepared ground surfaces. Surface preparations to make the ground less permeable include compaction and chemical treatment, or covering with impermeable materials such as plastic, rubber, corrugated iron, bitumen or concrete. In the case of ground-level rainwater harvesting, the storage tank will normally need to be located underground. The catchment area should also be protected (fenced off) to minimise the risk of possible faecal contamination.

The average quantity of water available from a rainwater catchment area is found by multiplying the area (in plan) with the mean annual rainfall in that area, and adjusting by an efficiency factor (average rainwater (litres) = catchment area (m^2) x mean rainfall (mm) x efficiency, where efficiency, has a value between 0 and 1,0). For roofs an efficiency of 0,8 is usual.

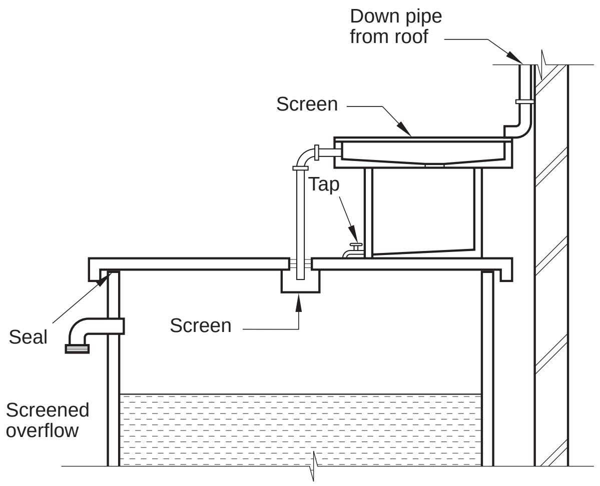


Figure 9.5: Arrangement for diverting the "first foul flush" (IRC 1980)

Fog harvesting

Fog harvesting is limited in application and confined to particular geographical areas. A number of pilot projects are currently being undertaken at Cape Columbine, Pampoenvlei, Lamberts Bay, Brand se Baai, Kalkbaken se Kop and Kleinsee. The publication *Fog harvesting along the west coast of South Africa: a feasibility study* by J. Olivier (Water SA, vol 28 no 4, October 2000) can be referred to in this regard.

The technique is fairly simple – nets spanned between poles. Fog condenses on the nets and runs down into an open chute for collection in a storage facility.

The findings of a number of test sites are currently awaited.

Surface water

The Catchment Management Agency or the Department of Water Affairs and Forestry's Regional Office should be consulted where surface water is used as source. Surface water sources, such as streams, rivers, lakes, pans and dams, will always contain suspended solids (turbidity) and microbiological pollutants. In addition, the quantity of water that can be abstracted from these sources is dependent upon droughts and floods, unless sufficient storage is available or can be provided. The following aspects should therefore be taken into account when relying on surface water to supply a community:

- The water should be treated for the removal or destruction of pathogenic organisms (e.g. bacteria, viruses, protozoa), as well as for turbidity.
- Where deemed necessary, a back-up source (e.g. a borehole) should be provided for times of shortage and drought, to ensure a minimum supply for domestic use.
- A pump station or other water extraction facility should be protected from possible damage by floods or vandalism.

The water supply intake may be sited at any point where the surface water can be withdrawn in sufficient quantities. In some situations where the gradient is steep enough, the water to be used may be diverted directly into a canal or pipeline, without the need for pumps.

In the case of a small stream or river it may be necessary to construct a weir across the river bed to provide enough depth for intake and to maintain the water level within a fairly narrow range. A weir may be constructed with concrete, cement blocks, or rocks covered with impermeable plastic sheeting. The type of construction selected will depend on economics and on the flood conditions expected.

The river or dam's intake point should be selected to abstract the best quality of water from the source. For example, a float intake (see Figure 9.6b) may be selected to withdraw water just below the surface. This may be desirable as the surface water may be clearer than the water at deeper levels. Alternatively, an intake placed below the bed of a river (see Figure 9.6d) would result in the water being partially filtered as it passes through the sand of the bed. While this may appear to be the most desirable, it is important to ensure that any such filtered-intake system is firmly fixed in place because, when the river floods, the river bed tends to become unstable.

In a stationary body of water like a dam or lake, it may be desirable to withdraw water well below the surface to minimise the amount of algae in the water extracted. However, if the water is extracted from too deep a level, the quality of this water may show a marked difference from the surface water. This is because of the possible thermal stratification of the lake in the warm summer months, when the oxygen levels in the deeper waters could be depleted, causing deterioration in quality.

Bulk-supply pipelines

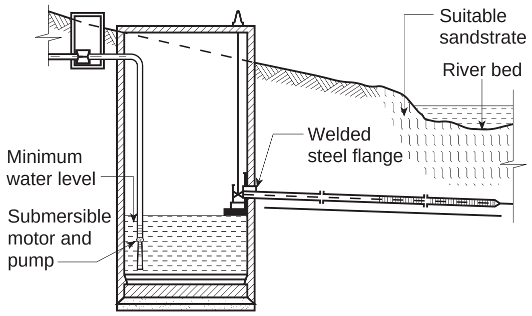
The storage and distribution system, often comprising the major expense, must be appropriate for the area in terms of cost, complexity and operational requirements.

When a developing area is located alongside an already developed area, it may be possible to purchase water directly from the authority supplying water to the developed area. In many cases, the existing water pipelines will be able to support the additional requirement of the developing area. If the pipeline is situated close to the developing area, this position could be highly cost-effective. A storage reservoir may be required to ensure a continuous supply where excess water is only available during off-peak periods. If the water in the pipeline is untreated, some treatment will be required to ensure that the water is safe for domestic use. The authority responsible for the pipeline will require payment for the water withdrawn from the pipeline, and hence it will be necessary to meter the connection.

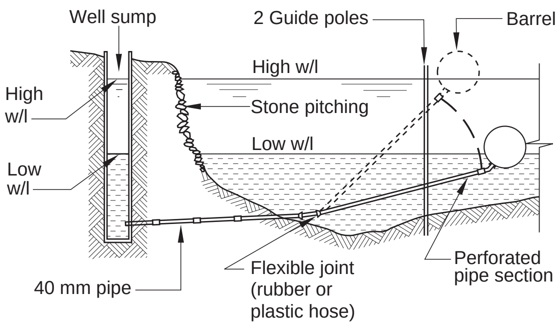
WATER TREATMENT

General

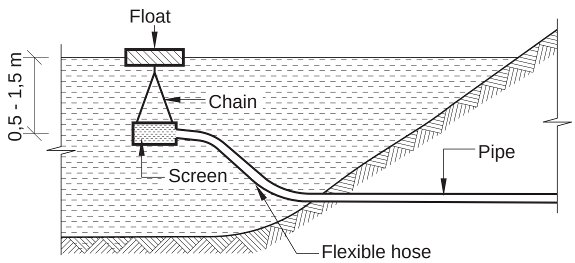
Water treatment is considered a specialised subject. This section therefore gives a broad background only and does not attempt to give guidance to the design engineer. Specialists should be consulted where water purification is considered.



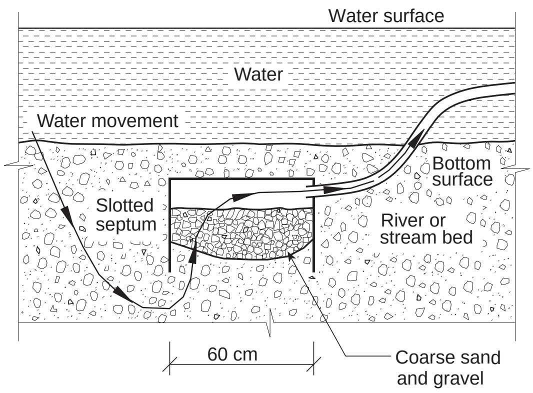
(a) River bank intake using infiltration drains



(b) Float intake



(c) Simple water intake structure



(d) The S.W.S. filter

Figure 9.6: Various surface water intake configurations

In many cases, water obtained from a particular source will require some treatment before being distributed for domestic use. Water obtained from boreholes, protected wells, protected springs and harvested rainfall often requires little or no treatment. However, as a precautionary measure and to minimise biological activity in the storage reservoirs and pipelines, even such waters should be chlorinated before distribution.

Most surface waters will require treatment, both to remove turbidity and for disinfection.

Certain surface waters and groundwaters will require additional treatment for the removal of organic and/or inorganic contaminants. Many groundwaters in southern Africa are highly saline and, unless a suitable alternative source of water can economically be located, they will require partial desalination to make them suitable for domestic use.

Unfortunately, there is no such thing as a universal, simple and reliable water treatment process suitable for small community water supplies. Treatment should be affordable and reliably operated. Okun and Schultz (1983) suggest that under all circumstances groundwater is the preferred choice for community supplies, as it generally does not require treatment. When treatment is required, this will be determined by the extent of contamination and by the characteristics of the raw water.

A simple approach for the selection of a treatment system is given by Thanh and Hettiaratchi (1982) (see Table 9.5). The emphasis on slow sand filtration is valid for areas where skilled personnel may not be permanently available to operate the plant, where chemical shortages may occur, where space is available at low cost, and where supervision may be irregular. Marx and Johannes (1988) found slow sand filtration to be an economical and successful option for water treatment plants in developing areas of South Africa.

Where sufficient money and skilled operators are available, standard water treatment plants (e.g. chemical flocculation, radial settler, rapid sand filtration and chlorination) have worked well under most circumstances.

Package water treatment plants

Package water treatment plants (see glossary) for smaller communities in rural areas have potential and could fulfil the need for potable water. Attention should, however, be given to operation and maintenance requirements as well as to backup from suppliers.

More information can be obtained from a Water Research Commission (1997) publication entitled *Package water treatment plant selection.*

Table 9.5: Treatment selection criteria (Thanh and Hettiaratchi 1982)

RAW WATER QUALITY	TREATMENT SUGGESTED
Turbidity 0-5 NTU Faecal coliform 0/100 mℓ Guinea worm or schistosomiasis not endemic	No treatment
Turbidity 0-5 NTU Faecal coliform 0/100 mℓ Guinea worm or schistosomiasis endemic	Slow sand filtration
Turbidity 0-20 NTU Faecal coliform 1-500/100 mℓ	Slow sand filtration Chlorination if possible
Turbidity 20-30 NTU Up to 30 NTU for a few days only Faecal coliform 1-500/100 mℓ	Pre-treatment advantageous Slow sand filtration Chlorination if possible
Turbidity 20-30 NTU Up to 30 NTU for several weeks Faecal coliform 1-500/100 mℓ	Pre-treatment advisable Slow sand filtration Chlorination if possible
Turbidity 20-150 NTU Faecal coliform 500-5 000/100 mℓ	Pre-treatment Slow sand filtration Chlorination if possible
Turbidity 30-150 NTU Faecal coliform >5 000/100 mℓ	Pre-treatment Slow sand filtration Chlorination
Turbidity >150 NTU	Detailed investigation (and possible pilot-plant study)

WATER SUPPLY OPTIONS

Selection of water supply terminals

Water supply terminals are divided into public (or communal) and private installations. Public or communal installations are those installations to which the public and the community have access. Private installations are those that render water to individual households.

The selection of terminals for a community depends on a number of factors, the most important being

- affordability of the system (by agency/users);
- selected method of cost recovery;
- unit cost to end-user; and
- long-term maintenance requirements.

With regard to water for domestic use, the relative importance of these factors for each terminal is given in Table 9.6. The value judgements in this table are subjective and a number of other factors may influence the final selection, or the validity of the judgement for a particular situation.

If possible, individual connections should be provided to schools, clinics and possibly some businesses, no matter which option is selected.

Public or communal water supply terminals

Rudimentary systems fall within the category of public or communal water supply terminals. These systems normally comprise a source or consumer terminal where water is collected in containers or buckets. Walking distance is usually between 200 and 500 metres. A minimum of water is provided – between 5

Table 9.6: Selection criteria for water supply terminals

TYPE OF SUPPLY	RELATIVE CAPITAL COST OF SUPPLY SCHEME	COST RECOVERY	MAINTENANCE NEEDS	UNIT COST
Public standpipes	medium	flat rate or per amount used	medium	low
Water kiosks	medium	per amount used	low	high
Tanker supply to tank	medium	flat rate	medium to high	medium to high
Vendors	low	per amount used	low	medium to high
Yard tank	low	per amount used	low	low
Roof tank	medium	per amount used	low	low
Metered yard tap	high	per amount used	low	low
Metered house connection	high	per amount used	low	low
Handpump	medium/low	flat rate	medium	medium
Spring supply to tank	low	flat rate	low	low

and 15 ℓ/c/d, mainly for drinking and cooking.

The most basic systems are run-of-river abstraction, rainwater harvesting, unprotected springs and open wells. The systems are often augmented by hand pumps, spring protection, a windmill, solar pump or some storage tanks. All systems require home treatment.

Communal street tap: Ordinary type

The system comprises a water reticulation system with standpipes in open areas or in road reserves. The Department of Water Affairs and Forestry's White Paper of 1994 defined basic water as access to 25 litres of potable water per person (capita) per day at a communal street tap which is within 200 metres of the dwelling, with 98% reliability and a 10 ℓ/min flow rate. Provision is usually made in the design for upgrading.

The design of the standpipe installation requires careful planning, and special attention should be given to drainage of excess water and avoiding wastage, in order to minimise health risks. A typical example is shown in Figure 9.7.

In the case of communal standpipes serving dwelling houses, the following criteria should be satisfied (payment arrangements may influence these considerations):

- one tap required per 25-50 dwellings;
- maximum number of people served per water point: 300;
- maximum number of people served per tap: 150;
- maximum walking distance from a dwelling to a standpipe: 200 m.

An acceptable discharge capacity from a standpipe is about 10 ℓ/min per tap. For commonly used taps the calculated discharge range, at an assumed efficiency of 80%, is given in Table 9.7.

These flow rates should be considered only as a guide; the actual flow rate depends on the type of tap used. The high discharge rates indicated for a 60 m head will normally be reduced by the limitations of the pipework. In practice, measured flow rates to single dwelling houses seldom exceed 40 ℓ/min.

In order to reduce water wastage, and to prolong the life of the tap washers, pressures should be limited.

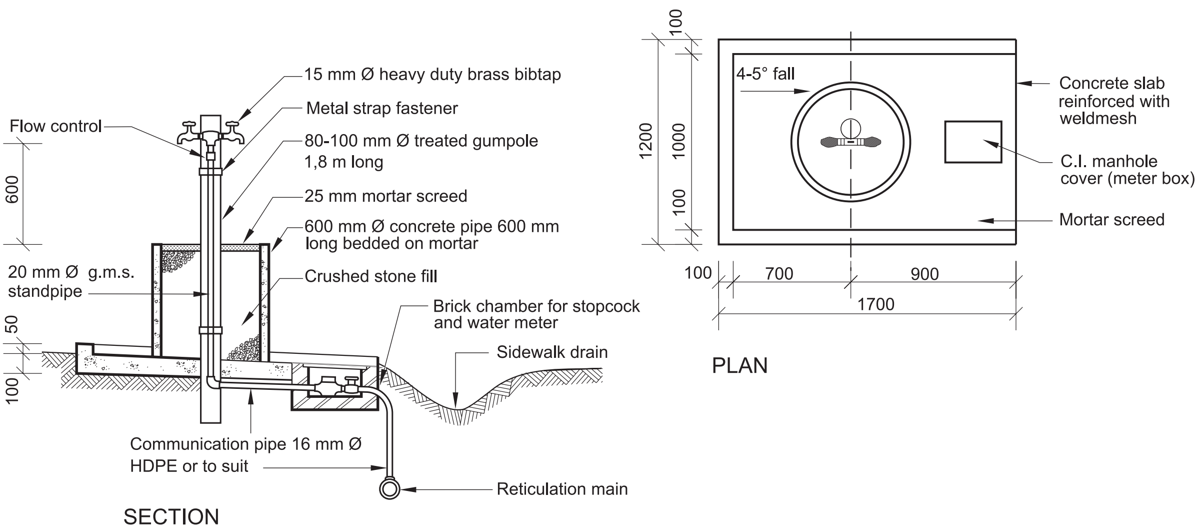


Figure 9.7: Typical standpipe detail

Table 9.7: Typical discharge rates for taps (assumed efficiency rate 80%)

TAP DIAMETER	DISCHARGE		
	5 m head	10 m head	60 m head
15 mm	16 ℓ/min	23 ℓ/min	54 ℓ/min
20 mm	22 ℓ/min	31 ℓ/min	70 ℓ/min

Access for physically disabled persons:
Barriers that prevent access to water and sanitation facilities for disabled people tend to fall into the following categories (WEDC, 2002):

- Environmental: barriers in the physical environment and infrastructure;
- Individual: functional limitations of the individual disabled person;
- Social: negative attitudes and behaviour of the community and society; and
- Institutional: discriminatory legislation, policies, and organisational practices.

Involvement of disabled people at all stages of project planning and implementation will improve both effectiveness and sustainability.

The provision of communal street taps should take cognisance of the requirements of physically disabled persons. Taps should be situated on smooth, even pathways, and ramps should be provided in lieu of steps. Ramps should be not less than 1100 mm wide with a slope not exceeding 1:12.

Communal street tap: Prepaid type

The basic (RDP) standard (25 ℓ/c/d within 200m at 98% reliability and 10 ℓ/min flow), with the addition of prepaid meters at street taps.

Water kiosks

Water kiosks are being used in developing areas where urbanisation has caused the rapid growth of settlements. The sale of water at kiosks provides an effective means of recovering costs, which is especially relevant in places where community management structures are not yet in place.

Due to their higher cost and the relatively large number of users required to make individual units commercially viable, kiosks are usually spaced further apart than standpipes would normally be. For the system to be viable, individual kiosks should supply at least 100 dwellings.

Facilities for accurately measuring and dispensing the standard purchase volume (usually 20 to 25 litres) should be provided. The structures should be sturdy, and have lockable facilities.

Water tanks with taps

Water tanks with taps may be the first level of supply improvement before any distribution piping is installed. Water may be supplied by gravity flow from a spring, from a borehole equipped with an engine-driven pump, by rainwater, or from a small treatment plant. People may need to walk long distances to the tanks to collect this water. However, its quality is usually good, and it may often be the only source of water available to a community.

The size and design of the tanks should be in accordance with the design principles given elsewhere in this chapter.

Handpump installations

The following criteria are considered important for effective water supply to areas using handpumps:

- The handpump chosen should take into account corrosiveness of the environment, together with suitable riser pipe material for the pumping head.
- The community should be consulted in the choice of the handpump; this selection should be made from a well-informed position.
- The pump should be installed professionally.
- The site should also be free from contamination by animals and humans, and generally be distant from sources of possible pollution.
- The pump should be taken care of by selected, motivated community members in order to facilitate maintenance tasks.
- It should not be expected of communities to be completely self-sufficient; adequate spares and maintenance should be available.

Private water supply terminals

Yard connections and house connections fall within the category of private water supply terminals.

Yard connections

Ordinary type
Water is provided, at pressure, at a tap on the boundary just within the stand. No storage facilities are provided on site and there is no supply to the house. However, an outside toilet may be supplied with a hand-washing facility or washtub.

Yard tank (or ground tank): low-pressure, trickle feed
Water is provided at full pressure up to specially manufactured yard tanks. The tank inlet has a flow regulator (trickle feed), which is sized to give a predetermined volume (mostly 25 ℓ/c/d) to the household. A ball valve inside the tank prevents it from overflowing. Supply can be shut off if required.

Yard tank (or ground tank): low-pressure, manually operated
This is similar to the Durban tank (Figure 9.8). Water is provided at full pressure into yard tanks. The volume of supply is either manually controlled by a bailiff who, on a daily basis, opens the supply at a control node, or electronically controlled to fill each tank where the monthly flat rate has been paid. The supply volume can be adjusted by changing the tank size and increasing the monthly flat rate. A ball valve inside the tank prevents it from overflowing. Supply can be shut off if required.

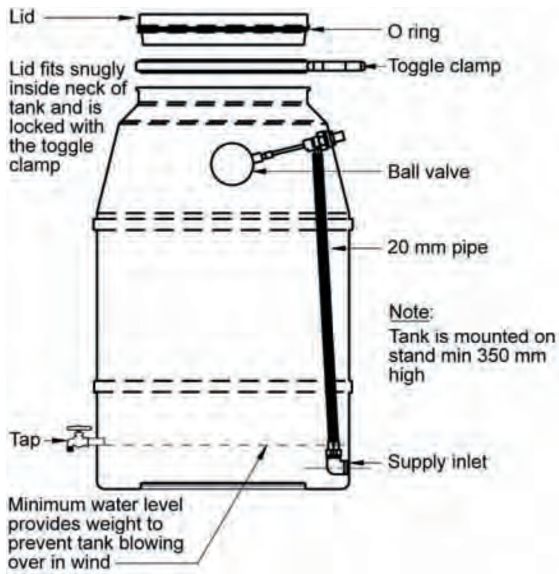


Figure 9.8: The Durban yard tank

Yard tank (or ground tank): low-pressure, regulated
Water is provided at regulated pressure and flow rate into yard tanks. Volume and pressure of supply is regulated by "equity" valves at control nodes located on the RDP pipe network. A ball valve inside the tank prevents it from overflowing. Bailiffs can shut off supply if required. Volume of supply can easily be adjusted by changing the size of the equity valve.

Roof tank: medium-pressure, manually operated
Water is provided at full pressure into a roof tank either in or on top of the roof of the house. The water supply is sometimes throttled to discourage excessive use. Water use in the house is at roof-height pressure. A ball valve inside the tank prevents it from overflowing. The volume of supply is unlimited and metered conventionally.

Customers are regularly invoiced for water consumed.

Roof tank: medium-pressured, regulated
Water is provided at high pressure to reticulation nodes, and at reduced pressure into a roof tank either in or on top of the house. The water supply is sometimes throttled to discourage excessive use. Water used in the house is at roof-height pressure.

The volume of supply is regulated by the "equity" valve at the node, and can be changed to upgrade or downgrade the level of water use. The volume of supply is unlimited and metered conventionally. Customers are regularly invoiced for water consumed.

House connections

Full-pressure conventional house connection
Water is provided at high pressure into the house, and all water use is at full pressure and unregulated flow. Water use is metered conventionally. Customers are regularly invoiced for water consumed.

The communication pipes for erf connections for dwelling houses (Residential zone 1) should be sized according to Tables 9.8 and 9.9.

For developments other than dwelling units metered individually, the communication pipe should be sized according to the specific demand.

House connection: full-pressure, prepaid
Water is provided at high pressure into the house, and all water use is at full pressure, and available with prior payment (prepayment tokens) activating the prepayment meter. These tokens can be bought at central vending offices. No monthly meter reading and billing is required.

DESIGN CRITERIA FOR WATER DISTRIBUTION AND STORAGE SYSTEMS

General

Water distribution and storage are, in most instances, the most costly parts of a water supply scheme. Hence savings in these areas through good design can often result in significant savings for a whole project.

The elements of a water distribution and -storage system include some or all of the following:

- bulk water transmission systems;
- bulk-storage reservoirs;
- intermediate-storage reservoirs;
- distribution networks; and
- terminal consumer installations.

This section deals with water demand and is presented in two parts. The first part deals with water demand in *developing areas* and the second part deals with *developed areas*. It does not therefore mean that the guidelines given are applicable only to a particular area – the designer should always be aware of the dynamics within a community that could influence the development of an area.

Table 9.8: Communication pipes across roads for house connections

INCOME LEVEL	MINIMUM ACTUAL INTERNAL DIAMETER (mm)	
	SERVING TWO ERVEN	SERVING ONE ERF
Higher	40, branching to 2 x 20	25, reducing to 20 at erf
Middle	40, branching to 2 x 20	25, reducing to 20 at erf
Lower	20, branching to 2 x 15	15

Table 9.9: Communication pipes on near side of road for house connections

INCOME LEVEL	MINIMUM ACTUAL INTERNAL DIAMETER (mm)	
	SERVING TWO ERVEN	SERVING ONE ERF
Higher	40, branching to 2 x 20	20
Middle	40, branching to 2 x 20	20
Lower	20, branching to 2 x 15*	15*

* The communication pipe may be reduced to 15 mm nominal diameter, provided the minimum head in the reticulation main at the take-off point for the erf connection under instantaneous peak demand is not less than 30 m.

Developing areas are considered to be those areas where the level of services to be installed may be subject to future upgrading to a higher level.

Developed areas are considered to be those areas where the services installed are already at their highest level and will therefore not require future upgrading.

Water demand

Water demand is usually based on historical consumption. Where water consumption records are not available, present consumption per capita can be estimated by consulting the residents. However, once the supply system has been upgraded, consumption is likely to change and Tables 9.10 and 9.11 may be used to estimate typical consumption. An improved estimate could be obtained by studying existing water supply systems in the same area. It has also been shown that extensive education programmes could have a positive influence on water demand.

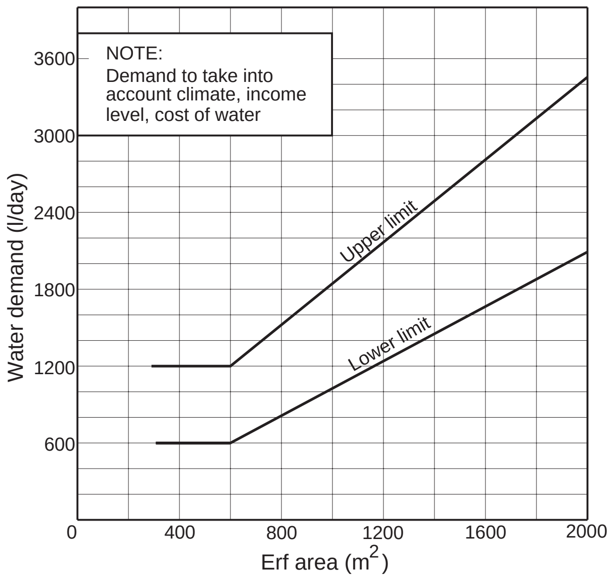


Figure 9.9: Annual average daily water demand for erven in developed areas

Table 9.10: Water demand for developing areas (IRC 1980)		
TYPE OF WATER SUPPLY	TYPICAL CONSUMPTION (ℓ/c/d)	RANGE ℓ/c/d
Communal water point		
• well or standpipe at considerable distance (>1000 m)	7	5-10
• well or standpipe at medium distance (250-1000 m)	12	10-15
• well nearby (<250 m)	20	15-25

Table 9.11: Water consumption in areas equipped with standpipes, yard connections and house connections (adapted from Department of Water Affairs & Forestry, (1992): Guidelines for the selection of design criteria)		
DOMESTIC WATER CONSUMPTION		
TYPE OF WATER SUPPLY	TYPICAL CONSUMPTION (ℓ/c/d)	RANGE ℓ/c/d
Standpipe (200 m walking distance)	25*	10 - 50
Yard connection		50 - 100
With dry sanitation	55	30 - 60
With LOFLOs		45 - 75
With full-flush sanitation		60 - 100
House connection (developed areas) #		60 - 475
Development level: Moderate	80	48 - 98
Moderate to high	130	80 - 145
High	250	130 - 280
Very high	450	260 - 480

* This consumption of 25 ℓ/c/d is the minimum to be made available per person in terms of government policy.
# The water demand in this category, based on a different approach, is also given in Table 9.14 and Figure 9.9.

Notes:

- A handpump should be considered as being similar to a well, since additional effort is required to obtain the water.
- A spring should be considered as being similar to a standpipe, especially when it has been protected.
- A climb of more than 60 m over a short distance should be considered as being similar to walking a distance of about 1 000 m.

Factors influencing water demand

- The type of sanitation system and the development level affect the water consumption, particularly in the category "House connection".

  The development levels are as follows:

  - *Moderate*: medium-sized formal housing with limited finishing, moderate gardens.
  - *Moderate to high*: limited formal suburban housing, moderate finishing, extensive gardens.
  - *High*: extensive formal suburban housing, small stands, extensive gardens, moderate-flush toilets.
  - *Very high*: formal suburban housing, large stands, extensive gardens, fully reticulated.
- Inhibiting factors like topography, water quality and water tariff.
- Metering and cost-recovery mechanisms.

Non-domestic water demand in developing areas

Water requirements for non-domestic purposes are difficult to estimate and, where possible, field measurements should be taken. Provision must also be made for the water demand at public open spaces. See Table 9.12 and 9.14 category 13.

Water demand for stock

The water demand for stock is given in Table 9.13. It must be pointed out that the provision of potable water for stock is highly undesirable and has serious cost-recovery implications.

Water demand in developed areas

The water demand figures in Table 9.14 should be used for detail design where applicable. Studies on water demand show there is usually a large degree of variance about the mean demand, and several factors can cause short-term variations. Therefore, the differences between Tables 9.11 and 9.14 should not be seen as significant and adjustments can be made to these demand figures to take local conditions into account. Attempts to base water demand on other erf-related data have not yet been validated.

Table 9.12: Non-domestic water demand

NON-DOMESTIC USERS	WATER DEMAND
Schools: day boarding	15-20 90-140 litres/pupil/day
Hospitals	220-300 litres/bed/day
Clinics	5 - outpatients 40-60 - in-patients litres/bed/day
Bus stations	15 - for those persons outside the community litres/user/day
Community halls/ restaurants	65-90 litres/seat/day

Table 9.13: Water demand for stock

STOCK	WATER DEMAND litres/head/day
Intensive: meat: LS	50
SS	12
Dairy: LS	120
Extensive: LS	50
SS	10

LS refers to large stock.
SS refers to small stock.
Intensive: the business of the land owner is farming.
Extensive: keeping a few animals for domestic purposes.

Table 9.14: Water demand for developed areas

CATEGORY	TYPE OF DEVELOPMENT	UNIT	ANNUAL AVERAGE WATER DEMAND (ℓ/day) UNLESS OTHERWISE STATED
1	Dwelling houses *(Residential zone I)*	Erf area for dwelling house	See Figure 9.9 for erven not exceeding 2 000 m^2. For erven >2 000 m^2, base demand on local conditions
2	Low-rise multiple-dwelling unit buildings *(Residential zones II and III)*	Dwelling	Upper limit 1 000[a] Lower limit 600[a]
3	High-rise multiple-dwelling unit buildings *(Residential zone IV)*	Dwelling	Upper limit 700[a] Lower limit 450[a]
4	Offices and shops	100 m^2 of gross floor area[b]	400
5	Government and municipal	100 m^2 of gross floor area[b]	400
6	Clinic	100 m^2 of gross floor area[b]	500
7	Church	Erf	2 000
8	Hostels	Occupant	150
9	Developed parks	Hectare of erf area	≤2 ha: 15 kℓ[c][d] >2 ha ≤10 ha: 12,5 kℓ[c][d >10 ha: 10 kℓ[c][d]
10	Day school / créche	Hectare of erf area	As per developed parks[d]
11	Boarding school	Hectare of erf area plus boarders	As per developed parks plus 150 ℓ/boarder
12	Sportsground	Hectare of erf area	As per developed parks
13	Public open spaces[e]	Hectare of erf area	See note [e] below

(a) Water demand includes garden watering of all common areas outside the limits of the buildings.

(b) Gross floor area obtained using applicable floor space ratio from the town planning scheme.

(c) Demand for developed parks to be considered as drawn over six hours on any particular day in order to obtain the peak demand.

(d) Where the designer anticipates the development of parks and sportsgrounds to be of a high standard, e.g. 25 mm of water applied per week, the annual average water demand should be taken as follows: ≤2 ha: 50 kℓ[c]; >2 ha ≤10 ha: 40 kℓ[c]; >10 ha: 30 kℓ[c].

(e) Refer to Chapter 5 and distinguish between "soft open space" and "semi-public open space".

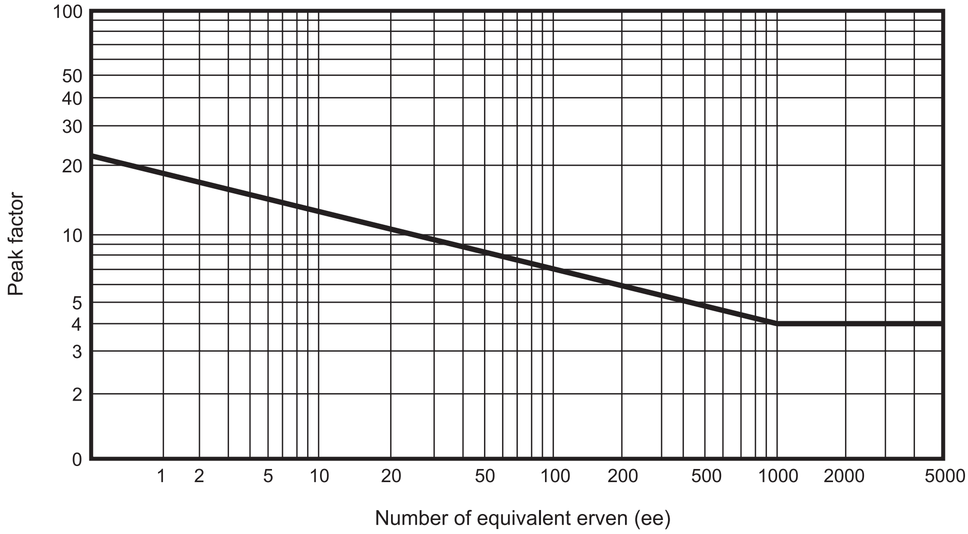


Figure 9.10: Factor for obtaining the peak flow in mains for low-cost housing, incorporating individual on-site storage

Peak factors

Figure 9.10 could be used as a guide where yard tanks are supplied and a single 15 mm tap is fitted on the service pipe between the consumer connection and the storage tank (usually a 200 litre capacity).

The peak factors mentioned in Table 9.15 are intended as a guide only. The actual choice of the peak factor requires considerable thought from the designer, and depends on several factors that must be taken into account. Recent studies have indicated that the peak factors currently in use are conservative; however, a comprehensive review is still outstanding.

The following are some of the factors that may significantly influence the choice of a specific peak factor:

- employment trends and practices in the community;
- gardening activities;
- number of persons per tap;
- agricultural activities;
- number of dwellings (where supply to less than 200 dwellings is being considered, consideration should be given to a higher peak factor; Figure 9.10 could be used as a guideline in this case);
- economic status;
- extent of unauthorised connections;

Table 9.15: Peak factors for developing areas

PEAK FACTORS: DEVELOPING AREAS – UNRESTRICTED FLOW SYSTEMS #				
TYPE OF DOMESTIC SUPPLY	SUMMER PEAK FACTOR	DAILY PEAK FACTOR	INSTANTANEOUS PEAK #	
			Low density**	High density**
House connection	1,5	2,4	3,6 - 4,0	4,0 minimum*
Yard connection	1,35	2,6	3,5 - 4,0	4,0 minimum*
Street tap / standpipe	1,2	3,0	3 - 3,6	4,0 minimum*
Yard tanks	-	-	see note	see note

\# Unrestricted systems are those systems where no specific arrangements restrict the flow at all. The instantaneous peak factor for restricted flow systems (yard tanks) is 1,5 at all times.

** Low-density areas are typically found in rural localities. High-density areas are those areas typically found in urban localities.

* Increases with diminishing number of consumers. Figure 9.10 could be used as a guide.

- "skeletonising" (see Glossary) of reticulation networks; in the case of low-level serviced communities, due consideration should be given to the design of the network for the ultimate scenario and the reticulation network being skeletonised to the requirements of the immediate level of service; and
- system constraints (e.g. maximum possible flow from a tap, or limited supply by a water bailiff). Reticulations in the developing areas may be significantly affected by the discharge rate from standpipes. Table 9.7 gives guidelines on tap discharges.

It is not advisable to reduce peak factors to effect cost savings. Cognisance must be taken of the fact that the water supply to developing areas could be upgraded at a later stage, and the long-term development of the water supply to the community must be taken into account.

Peak factors for developed areas

In order to determine the instantaneous peak factor for developed areas from the graph, the type of development should first be converted to "equivalent erven" (ee) according to the design annual average daily demand, accepting as a basis for design that one ee has an annual average daily demand of 1 000 litres.

Using the ee thus obtained, the instantaneous peak factor pertaining to any point in the network should be obtained from Figure 9.11.

The annual average daily demand multiplied by the peak factor gives the instantaneous peak flow.

Storage

The peak factor will be reduced in the case of the provision of terminal storage (in which case only the terminal storage volume is designed to cater for peak demands).

The provision of intermediate storage will also result in a reduction in the peak flows in the elements prior to the intermediate storage facility.

Where the installed capacity is unable to cater for the peak demand, the demand curve will flatten out, resulting in the actual peak demand being limited to the supply capacity of the system, extended over a longer period of time. This may be of some inconvenience to residents, but will not lead to a water shortage.

Residual pressures in developing and developed areas

To obtain the residual head at any point in the reticulation, the network should be balanced using instantaneous peak flows and fire flows.

Hydraulic formulae for sizing components

Any of the recognised hydraulic formulae may be used to size pipeline components.

WATER TRANSMISSION

General

Pipelines are the most common means of transmitting water, but canals, aqueducts and tunnels may also be used. Water transmission conduits usually require considerable capital investment and therefore all technical options with their associated costs should be carefully evaluated when selecting the best solution in each particular case.

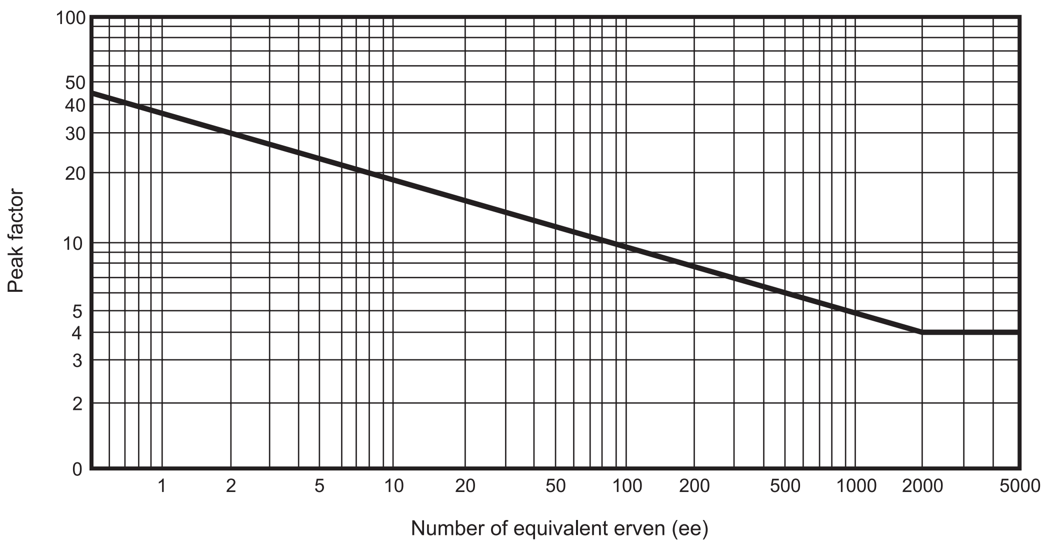


Figure 9.11: Factor for obtaining the peak flow in mains in developed areas

Pipelines will usually mean minimum water losses, and also imply the shortest transmission distance. However, pipeline costs may be considerable and the option of canals for the transportation of non-potable water could be considered. Canals will result in higher losses, longer transmission distances, and the possibility of deterioration in water quality due to algal growth. However, the lower costs and the option of labour-intensive construction may make this option more attractive. Designers should also take into account the habits and lifestyle (keeping of livestock) of those communities where canal systems for water supply are considered.

Water can be transported either by gravity or pumping, or a combination of these. Clearly, the preferred choice will always be a gravity supply. Aqueducts and tunnels should only be used in special circumstances. However, physical or economic constraints may limit the options and necessitate a pumping component.

Canals

Canals may be used to transport large volumes of water over long distances. They should be lined (usually with concrete) and inspected regularly for cracks and leaks in the joints. The growth of algae may need to be addressed by shock chlorination from time to time. Canals should only be used for transporting non-potable water.

Water tankers

The operational costs of supplying water by tanker are usually extremely high, but may serve as a temporary measure in an emergency situation, or for a new settlement. However, alternative tank size and delivery vehicle combinations should be considered when undertaking a feasibility study. The use of multi-purpose vehicles (e.g. tractors with different trailer combinations) could also be considered as a means of reducing the capital costs.

PIPELINE DESIGN

Basic requirements

- The static pressure should be kept as low as possible by reducing the pressure in a balancing or separate break-pressure tank, or by means of a pressure-reducing valve.
- To avoid air pockets, the number of high and low points should be kept to a minimum by trying to follow the contour lines, rather than roads or tracks.
- To minimise the number of air-release valves, the pipeline trench depths may be varied to avoid local high and low points.
- The cost of a water transmission system is more sensitive to the total length of pipe installed than the diameter of the pipes. Therefore, it is generally advantageous to design a transmission system (at least the major components) to meet the ultimate capacity.
- Velocities in pipes should be approximately 0,6 m/s and should not exceed 1,2 m/s.
- Velocities in special fittings (fittings specifically manufactured) should not exceed 6 m/s.
- Air valves should be installed at summits, and scour valves at low points between summits.
- Thin-walled pipes susceptible to buckling must have valves that automatically allow air to enter when the pipeline is emptied, so as to prevent a vacuum that will cause the pipe to collapse.
- To facilitate the location of a buried pipe during maintenance, curved pipe routes should be avoided. All bends should be marked with a post or a suitable beacon, and the pipeline laid in a straight line between bends.
- To avoid air pockets in pipelines, the slope should be greater than 0,3% (0,3 m per 100 m length), or 0,2% for large-diameter pipes (>200 mm).
- To avoid damage to pipelines during backfilling of a trench, a proper pipe-laying specification should be provided by the designer. Recommended minimum trench depths are as follows:
  - *road crossings*: pipe diameter + bedding + 0,80 m;
  - *otherwise*: pipe diameter + bedding + 0,60 m.
- Bedding thickness should normally be a minimum of 0,10 m or one-sixth of the pipe diameter, whichever is greater.
- The likely effect of water hammer/surge pressure should be considered in the design of a pressure pipeline system, as this may be the crucial factor in the selection of pipe class, or may indicate the need to provide surge or accumulator tanks.

The suitability of any pipe for a particular application is influenced by:

- its availability on the market, both in respect of dimensions and pressure classes;

- its purchase price and associated costs of valves and fittings;
- its susceptibility to corrosion, mechanical damage and material ageing, as well as any other cause of material deterioration in the particular application;
- its storage costs; and
- the diameter of pipeline (internal and external).

Pipes laid above ground

It may happen that pipes have to be laid above ground due to adverse conditions, such as rock. When considering laying pipes above ground, the following factors need to be taken into account:

- Provision should be made for expansion joints (the effects of thermal changes must be considered).
- Each pipe section should be properly supported. The supports should be designed to carry the load of the pipe as well as the water it conveys. Sufficient supports should be included in order to prevent sagging of the pipeline. The pipe should be strapped to the support, leaving room for movement.
- Adequate thrust blocks should be designed to cater for hydraulic forces at bends.
- Adequate anchoring should be provided, especially in steep slopes, and directional and elevation changes.
- The consequences of pipe failure should be evaluated.
- Heavy equipment like valves should be supported independently.
- Adequate protection should be provided to cope with external abrasion.
- Vulnerability to damage from veld fires, animals and rodents should be assessed.

VALVES AND OTHER FITTINGS

General

Valves and fittings should be carefully located and designed to facilitate the operation of the system. Careful routing of the pipeline will minimise the number of costly fittings required.

Isolating valves

Isolating valves should be installed at one- or two-kilometre intervals on transmission mains. Where possible, these should be combined with air-release valves. Pressure-relief valves should be installed on pumping mains to avoid damage caused by pumping against closed valves.

In reticulation networks, isolating valves should be provided so that not more than four valves need to be closed to isolate a section of main. Valves should be spaced so that the total length of main included in an isolated section does not exceed a nominal 600 m. This will obviously depend on circumstances.

In order to facilitate identification, valves should be located at street corners opposite erf corner boundary (splay) pegs, and intermediate valves opposite the common boundary peg for two erven.

Where pipes intersect, isolating valves should generally be installed in the smaller-diameter branches.

Isolating valves should be installed to facilitate maintenance of the main, and generally located to suit the topography.

To reduce cost, isolating valves in larger mains may be of lesser size than the pipeline. The design should ensure that the cost of the smaller valve, together with reducers, is less than the cost of a full-size valve. In addition, care should be taken to ensure that velocities through the valve are not excessive.

Depending on the size of the valve and the unbalanced pressure across the valve, devices such as thrust bearings, spur gearing and a separate bypass valve may be required.

When flanged isolating valves are used, a flange adaptor coupling should be installed to facilitate removal of the valve.

Air valves

Air valves are required to release air from the pipeline during the filling process and during normal operation. Whereas automatic small-orifice air-release valves are desirable, these may be replaced with public standpipes or other suitable distribution points. As air-release valves require servicing from time to time, it is recommended that a gate valve be installed with the air-release valve for easy removal and repair.

Where possible, pipelines should be laid such that the need for air valves is avoided. Fire hydrants can also be used to vent the main during charging.

Air valves are a possible source of contamination, and the air-valve installation arrangements should be such that contamination of the system cannot occur, while an adequate air flow for the valve is always maintained.

Air valves should be provided to suit the longitudinal section of the pipeline in relation to the hydraulic gradient.

Air valves should be sized according to the air flow rate generated by the rate of inflow or outflow of the water in the pipeline.

Air valves should be installed with an isolating valve on the air valve branch to facilitate maintenance, and should preferably be fitted with a cock tapped into the bottom of the valve body, to enable the effective operation of the valve to be checked.

Scour valves and outlets

Scour valves should be installed at low points in pipelines with a diameter of 80 mm or more. A scour valve comprises a hand-operated valve on a drainpipe of a diameter 0,4 to 0,6 times the diameter of the pipe being drained. There should be an open drain to lead the washout water to a suitable watercourse.

Scour outlets not connected to a stormwater drain system should be designed to limit the erosion caused by the escaping water.

In reticulation networks, a fire hydrant should, if possible, be positioned so that it can be used as a scour valve. Dead-end mains should terminate in a scour point.

Scour outlets should be sized to permit complete draining of a section of main between isolating valves within two hours.

Anti-vacuum valves

Anti-vacuum (or air-admission) valves should be provided downstream of each section valve in a transmission pipeline, to prevent the build-up of a vacuum when the section valve is being closed. Most air-release valves also act as air-admission valves.

Break-pressure devices

Break-pressure devices may be either break-pressure tanks or pressure-reducing valves.

Where possible, break-pressure tanks should be combined with balancing tanks.

When used, pressure-reducing valves should be provided with a pressure-relief valve on the outlet side, to prevent the possible build-up of pressure resulting from failure of the pressure-reducing valve to operate correctly. The discharge from the relief valve should be conspicuous when it occurs.

The installation should also be provided with a dirt box upstream of, and a bypass pipe around, the pressure-reducing valve, complete with an isolating valve protected against accidental opening. A pressure gauge should be provided on both the upstream and downstream sides of the dirt box.

Marker posts

Marker posts should be placed along the pipelines at intervals sufficient to facilitate location of the route. Marker posts should also be placed at all pipe bends, junctions, and other features.

Anchorage and thrust blocks

Anchorage and thrust blocks should be used whenever the pipeline changes vertical or horizontal direction by more than 10°. Thrust blocks should also be used where the size of the pipeline changes, at blank ends, and on steep slopes (more than 1:6).

Surge control

The likelihood of pressure surges should be investigated and, where necessary, provision made for surge control.

Valve chambers

Sufficient working space to allow a spanner to be used on all bolts should be provided in chambers for isolating, air and other valves.

Venting of air-valve chambers should allow for adequate air flow.

Roof slabs should be designed to allow for removal and replacement of the valve.

Valve chambers should, where possible, be finished proud of the final ground level.

Where necessary, the design should make provision for the possibility of differential settlement between the valve chamber and the pipeline.

WATER STORAGE

General

The purpose of storing water is to meet balancing requirements and cater for emergencies (e.g. fire-fighting) or planned shut-downs.

The balancing volume is required to cater for peak outflows while a constant (or variable) inlet flow is being received.

Reservoir storage

Where water is obtained from a bulk water supply authority, the storage capacity provided should comply with the requirements of the authority. A storage capacity of 48 hours of annual average daily demand is suggested, although there may be situations where 24 hours will suffice.

The nominal capacity for elevated storage, based on the typical period involved in power failures, is given in Table 9.16.

The nominal capacity of the duty pump should be equivalent to the sum of the instantaneous peak demand and the fire demand (obtained from the section on provision of water for fire-fighting), or the instantaneous peak demand plus an allowance of 20%, whichever is the greater.

All pumps should be rated for similar duties so that they are interchangeable.

The standby power source should operate automatically in the event of an electricity supply failure.

Under certain limited circumstances, as an alternative to providing elevated storage and pumps, a scheme comprising booster pumps (variable speed can also be considered) delivering directly into the reticulation can be considered. The total cost, including running and maintenance, should be taken into account. This type of scheme is, however, not a preferred option.

The capacity of the supply main to the reservoir should be designed to provide an inflow rate to the reservoir of not less than 1,5 x annual average daily demand for the area served by the reservoir.

Note should be taken of existing water consumption patterns that can be applied to the area to be served by the planned service reservoir. Demand patterns should lead the design engineer at all times.

It will be to the advantage of the local authority to make use of optimal design techniques for determining reservoir storage, like the time-simulation and the critical-period techniques. Both these methods were developed in the mid-1980s and make full use of the interrelationship between the feeder-main capacity and the reservoir capacity. The time-simulation technique is based on analysing river reservoir behaviour. The critical-period technique is based on the relationship between the total storage required and the feeder-main capacity. If use is made of the techniques mentioned, 48 hours' storage capacity need not be provided.

Caution should be exercised when determining the capacity of reservoirs – too large a reservoir may cause problems associated with stagnant water.

Other storage reservoirs

Storage reservoirs may also be required for the following primary purposes:

- water collection from various sources;
- to provide contact time for a certain water treatment operation, such as chlorination; and
- to provide water to a pump station (booster reservoir).

A reservoir may be either at ground level or elevated.

Location of service reservoirs

Where the main storage reservoir also serves as the service reservoir (i.e. it supplies water at the required pressure to the farthest point in the area), the reservoir should be located near the centre of the area. In flat areas this may be achieved by constructing an elevated tank at the centre. In undulating areas it will usually be more advantageous to select the highest point for its construction. By locating the tank as close to the centre as possible, distribution pipe costs can be minimised, and a more even distribution of pressure achieved.

Alternatively, the reservoir could be situated between

Table 9.16: Elevated storage capacity

PUMPING PLANT SERVING THE ELEVATED STORAGE FACILITY	CAPACITY OF ELEVATED STORAGE (HOURS OF INSTANTANEOUS PEAK DEMAND)*
One electrically driven duty pump, plus one identical electrically driven standby pump, plus standby power generation independent of the electricity supply.	2
One electrically driven duty pump, plus one identical electrically driven standby pump	4

* derived as described in the section above on peak factors.

the distribution area and the source of supply, or at the highest point surrounding the distribution area to obviate the need for elevated storage.

It is also advisable that, if possible, the difference in elevation of the highest water level in the storage reservoir and the lowest laid pipeline should not be more than 60-70 m. If this difference is greater it may be necessary to provide break-pressure devices in the distribution system.

Intermediate storage

The provision of intermediate storage can have a number of objectives, the most important of which are:

- a reduction in the sizes of the main distribution pipes, by reducing the peak-flow demand of these mains;
- a reduction in pipeline pressures;
- a reduction of the impact of supply breakdowns;
- a division of the supply into smaller subsections which can be more easily managed by community organisations; and
- a reduction in the size of the main storage reservoir, in terms of both balancing storage and emergency storage.

The provision of intermediate storage will usually be economically feasible only in areas where the topography is steep enough to obviate the need for elevated storage, or where undulating topography dictates the need for different pressure systems for different sections of the community.

The selection and design of intermediate storage will be based on criteria similar to that for bulk-storage reservoirs.

On-site storage is discussed in the section on "Terminal consumer installations" elsewhere in this chapter.

DISTRIBUTION NETWORKS

General

One of the most important requirements for an economic distribution system is the location of the service reservoir as near to the distribution network as possible. Among other advantages, peaks are evened out and fire protection can be more easily achieved.

General requirements for distribution network design

- The maximum head during the reticulation (under static conditions) should not exceed 90 m.
- The minimum head during the design peak flow should be according to Table 9.17.
- Minimum pipe sizes should not be less than 50 mm (internal diameter).
- Major reticulation pipes should be sized to suit the design period.
- It is advantageous to use the minimum number of different pipe sizes to reduce the holding stock required for maintenance and repair.
- Wherever possible, pipelines should be laid in road reserves and preferably not pass through residential or privately owned property.
- Pipelines on private land should be protected by servitudes in favour of the service owner.
- Where pipelines need to cross roads, care should be taken to ensure that the pipe is well bedded and at a sufficient depth (min 0,8 m cover). It is advisable to maintain a larger-diameter pipe for road crossings than may be required from design considerations.
- Comments in the section on valves and other fittings are also valid for distribution networks.

Table 9.17: Residual pressures

TYPES OF DEVELOPMENT	MINIMUM HEAD UNDER INSTANTANEOUS PEAK DEMAND (m)	MAXIMUM HEAD UNDER ZERO FLOW CONDITIONS (m)
Dwelling houses: house connections	24*	90
Dwelling houses: yard taps + yard tanks	10*	90

* Plus the height difference between the main and the highest ground level at any point on the erf not exceeding 50 m from the boundary adjacent to the main. A minimum head of 5 m for site-specific cases (e.g. developing on top of a hill) may be considered.

- Where air valves and scour valves are required, taps on standpipes or at other terminals should be sited to fulfil this function wherever possible.
- Isolating valves should be located at street corners, or opposite erf corner boundary pegs.
- Where pipes intersect, isolating valves should be installed in the smaller-diameter branches.
- Pipes should not rise above the hydraulic grade line.
- Flushing points (or fire hydrants) should be provided where dead-end mains cannot be avoided.
- A developing practice, "skeletonising" of the network, should be considered where developing areas' services are subject to future upgrading.

Residual pressures

The reticulation system should be designed so that the residual pressure in the reticulation main at any point is within the limits given in Table 9.17 (the desirable residual pressures applicable during fire-flow conditions are given in Table 9.20).

MATERIALS

Considerations in the selection of materials

Most of the materials referred to in these guidelines are listed and described in the relevant sections of the SABS 1200 series and SABS product specifications. Refer to product specifications for details of working pressures and dimensions of pipes made from the alternative materials. Specifications other than SABS should also be consulted where no applicable SABS specifications exist (e.g. ISO).

The materials suitable for use on a particular project and the internal and external corrosion-protection systems for the pipes, joints, fittings and specials may be specified by the controlling authority. If not, the following factors should be considered when selecting suitable materials:

- the life-cycle cost (initial capital plus maintenance costs);
- the chemical composition of the water to be distributed or stored (for example, certain types of pipes may not be advisable for conveying water with a Langelier Saturation Index of less than -0,5, and a detailed assessment of the circumstances will be necessary in such cases); brass fittings, couplings, valves, etc, particularly if soft water is conveyed, should be especially resistant to de-zincification;
- the corrosive nature of the soil and ground water, and the possible existence of stray electric currents; and
- the structural strength of the pipes and reservoirs.

Circumstances requiring particular attention are heaving clay soils, dolomitic areas and high external loading.

In the case of rigid pipes of small diameter, the designer should check for the possibility of beam-type failure. SABS Code of Practice 0102:1987 – Part 2 gives guidelines on the external loadings that can be applied to buried pipelines. The minimum class of pipe, from a structural consideration, should be Class 9.

Materials for pipelines

- Cast iron, steel and galvanised steel are the strongest pipe materials and should be used where high operating pressures are expected. The cost of fittings, especially at high pressures, and the susceptibility of these pipes to corrosion, should, however, be kept in mind. Joint types include threaded, Viking Johnson-type flexible couplings, continuously welded, flanged or spigot and socket types with rubber rings.
- Fibre-cement (asbestos-cement, FC) pipes cost less than iron and steel pipes. FC pipes should preferably be bitumen-dipped. Care must be exercised to ensure good bedding of the pipes when they are installed, as they are susceptible to fracturing as a result of ground movements. Couplings include asbestos-cement sleeves with rubber rings, cast-iron flexible couplings, or Viking Johnson-type flexible couplings. FC bends are not recommended.
- Unplasticised polyvinyl chloride (uPVC) and modified polyvinyl chloride (mPVC) provide easy-jointing pipes and good corrosion resistance. However, PVC pipes suffer a loss in strength when exposed to sunlight, and should therefore not be stored in the open. Pipes may be damaged by careless handling and, as with FC pipes, must be carefully bedded, avoiding stones and hard edges. The preferred type of coupling is spigot and socket rubber ring joint.
- Polyethylene (PE) pipes are supplied in rolls and are relatively flexible. Thus the number of joints and bends is greatly reduced. PE does not deteriorate significantly when exposed to sunlight. There are two types of polyethylene: low-density polyethylene and high-density polyethylene. Low density PE is mainly used for irrigation purposes. High-density PE is suitable for small-diameter mains, secondary pipelines and service pipes. Joints on larger diameter HDPE pipes are typically made

by butt-welding. On smaller pipe sizes, compression-type joints are used.

- Reinforced and pre-stressed concrete are suitable for long, large-diameter transmission lines. Both types have considerable strength and are resistant to corrosion. Spigot and socket joints are used for most reinforced concrete pipes. Pre-stressed pipe-jointing systems can vary, and depend to some extent on the pre-stressing design.

- Glass-reinforced polyester pipes have been made available on the market recently. Various projects have been completed successfully using these pipes. The pipes are also suitable for long and large transmission lines. No corrosion control is required and the pipe compares favourably with the PE pipe.

Materials for communication pipes

Materials generally suitable for communication pipes are:

- galvanised steel with screwed and socketed joints or Viking Johnson-type flexible couplings; and

- polypropylene (PP), high-density polyethylene (HDPE) and low-density polyethylene (LDPE) with external compression-type joints.

Where pipes are laid in aggressive soils, especially where moisture is retained in the soil under a paved surface, metallic pipes must be well protected against corrosion. Plastic pipes are more suitable under these conditions.

Materials for reservoirs

Large reservoirs (>200 m^3) are usually constructed of steel or reinforced concrete. Elevated tanks of steel panels have proved reasonably successful in areas away from the aggressive coastal environment. Lined earth reservoirs with floating covers, or concrete reservoirs with floating covers, are an economical choice for ground reservoirs in South Africa.

For smaller (<50 m^3) reservoirs, ferrocement, masonry, galvanised iron, asbestos-cement and certain plastic and rubber tanks may also be used. Polyethylene and fibreglass tanks, if used for potable water, should be constructed so as to prevent light penetration, which may encourage algal growth.

Traditional structures employed for water storage in the rural areas are not always durable or hygienic. It is often not possible or desirable to erect reinforced-concrete water-retaining structures in these remote areas because the cost may be prohibitive.

Ferrocement

The use of ferrocement tanks in rural areas may be considered, since their construction can be undertaken without sophisticated equipment or highly trained manpower.

Ferrocement simply consists of one part cement mixed with two parts sand and just sufficient water to form a paste-like consistency. This is forced onto layers of closely and evenly distributed wire mesh reinforcement (chicken netting) by hand or by trowel. This technique allows for complex structures with a thin shell thickness.

It has a high strength-to-mass ratio when compared to reinforced concrete. It also requires little or no maintenance when compared to metal tanks, and is more durable than fibre cement. Readers are referred to the following publications, which are noteworthy:

How to build a small ferrocement water tank: CSIR, Division of Water Technology (1988). ISBN 0 7988 34315.

Ferrocement water tanks: International Ferrocement Information Centre. (Available from the Cement and Concrete Institute, Midrand.)

Masonry

Masonry tanks have been constructed successfully and have been in use for many years. Standard plans are available from the Department of Public Works. This type of construction can also be executed with lesser skilled manpower.

Galvanised tanks

Galvanised tanks are also suitable for use in rural areas. Although these tanks can be erected in a short space of time, designers should consider their cost. Specialist contractors are usually employed and opportunities for training and the use of local manpower are therefore limited. Galvanised tanks are also notorious for poor thermal insulation and their service life, even with regular maintenance, is not comparable with, for instance, concrete reservoirs.

Asbestos cement, plastic, fibreglass, polyethylene and rubber tanks

The main advantage of these products is the speed involved in making available storage capacity. Once again, limited opportunities exist for use of local manpower. Transporting these prefabricated tanks is usually difficult. Polyethylene tanks should be considered as only a temporary measure.

CONSTRUCTION

General

Designers ought to be aware of the need for job creation in the provision of services and facilities, and should take this into account. It is also important to consider the requirements of government departments, municipalities and other service authorities in this regard. The decision as to whether specifications are to be modified in order to promote job creation lies with the employer and is not discussed in these guidelines.

National Standardised Specifications for Engineering Construction

The sections of SABS 1200 listed below are recommended for general use:

SABS 1200 A:	General
SABS 1200 AA:	General (Small works)
SABS 1200 D:	Earthworks
SABS 1200 DA:	Earthworks (Small works)
SABS 1200 DB:	Earthworks (Pipe trenches)
SABS 1200 G:	Concrete (Structural)
SABS 1200 GA:	Concrete (Small works)
SABS 1200 L:	Medium pressure pipelines
SABS 1200 LB:	Bedding (pipes)
SABS 1200 LF:	Erf connections
SABS 1200 LK:	Valves
SABS 1200 LN:	Steel pipe and linings

Watertightness test

For the requirements and tests for watertightness of reinforced concrete reservoirs and elevated storage facilities, refer to Clause 3.3.38 of SABS 0120:1980 – Part 2, Section G.

Disinfection of reservoirs and elevated storage facilities

Following completion of construction, the structure should be cleared of debris and the floor swept clean using damp sawdust, to prevent dust from rising. The walls and floors should be hosed down and the water drained away. Water should be admitted into the structure to a depth of approximately 300 mm and uniformly chlorinated so as to attain a minimum chlorine residual of 10 mg/ℓ.

All internal surfaces of the structure, including pipework, should be thoroughly hosed down with the chlorine solution. After all personnel have vacated the structure, a quantity of the chlorinated solution should be poured over the internal access ladder.

The chlorinated solution should be drained prior to filling the structure with potable water.

Should it be necessary for the structure to be emptied after the initial filling so that personnel can gain access to the water retaining portion of the structure, then the disinfection process described above should be repeated.

Markers for valves and hydrants

The positions of isolating valves and hydrants should be clearly indicated by means of permanent marker posts located on the verge opposite the fitting, or painted symbols on road or kerb surfaces.

Symbols on markers should be durable. Where services pass underneath national or provincial roads, markers should be placed on both sides of the road reserve. Similarly, markers should be placed on both sides of servitudes of other service providers.

MANAGEMENT OF WATER DISTRIBUTION SYSTEMS

General

It is not the purpose of this section to discuss all the functions of management as far as water supply systems are concerned. This section is primarily aimed at informing the engineer of the importance of the control of unaccounted-for water in supply systems, and other aspects relating to regulations in this regard. The Water Services Act requires that a Water Services Authority should have full control of water supply in its area of jurisdiction, and therefore unaccounted-for water will be an integral part of management reports.

Unaccounted-for water

The SABS 0306 Code of Practice should be followed in accounting for potable water within distribution systems and in the corrective action to reduce and control unaccounted-for water.

The code also gives guidance on the design of water reticulation systems with water management in mind, and gives guidance on leak detection and pipeline monitoring.

Metering

The information required for accounting and management is generated by proper metering. The importance of metering cannot be overemphasised. The Regulations relating to Compulsory National Standards and Measures to Conserve Water (Government Notice R 22355 dated 8 June 2001), published in terms of the Water Services Act, is a very important document that governs the relationship between the local authority and the consumer.

The regulations stipulate that water to any consumer must be measured by means of a water-volume-measuring device, and that all water be supplied in terms of an agreement between the local authority and the consumer. Metering water districts within water distribution schemes is also a requirement.

Those involved in water supply to communities should take note of these regulations.

Guidelines for metering can be found in the catalogues of meter suppliers.

It is important to note the following requirements:

- all mechanical meters must comply with SABS specifications;
- all meters must be installed according to the manufacturer's instructions;
- meters must be correctly sized;
- meters are to be tested (and replaced if necessary) at regular intervals;
- unmetered connections should not be allowed;
- regular inspection of actual flow is required to confirm meter sizes, where larger meters are installed;
- meter installations should at all times correspond with financial records;
- prepaid water meters should be considered if the community is in favour of this option;
- meters must comply with the Trade Metrology Act (Act 77 of 1973); and
- consumer installations must comply with SABS 0252: Water supply and Drainage for Buildings.

PROVISION OF WATER FOR FIRE-FIGHTING

General

The provision of water for fire-fighting should comply with the requirements as described in SABS Code of Practice 090:1972 – Community Protection against Fire, but with deviations from and additions to the code as described below.

Scope of the SABS Code of Practice 090:1972

The code covers recommendations for the organisation of fire services, water supplies for fire-fighting, and by-laws relative to fire protection. It includes a fire-rating schedule.

It is also intended for use by designers of water supply systems for normal industrial areas, central business districts and residential settlements. Specifically excluded from this document are special types of development, such as bulk oil and fuel storage facilities and airports. For these types of development, reference must be made to specific standards or regulations governing the fire-service requirements.

Fire-risk categories

Areas to be protected by a fire service should be classified according to the following fire-risk categories.

High-risk areas

These areas in which the risk of fire and of the spread of fire is high, such as congested industrial areas, congested commercial areas, warehouse districts, central business districts, and general residential areas with floor space ratio of 1,0 and greater where buildings are four storeys and more in height (residential zone IV).

Moderate-risk areas

These are areas in which the risk of fire and of spread of fire is moderate, such as industrial areas, areas zoned "general residential" with a floor space ratio of less than 1,0 (residential zones II and III) where buildings are not more than three storeys in height, and commercial areas normally occurring in residential districts where buildings are not more than three storeys in height.

Low-risk areas

These are areas in which the risk of fire and the spread of fire is low. This category is subdivided into four groups as follows.

- <u>Low-risk – group 1</u>: Residential areas (residential zone 1) where the gross floor area of the dwelling house, including outbuildings, is generally likely to be more than 200 m^2.
- <u>Low-risk – group 2</u>: Residential areas (residential zone 1) where the gross floor area of the dwelling house, including outbuildings, is likely to vary between 100 m^2 and 200 m^2.
- <u>Low-risk – group 3</u>: Residential areas (residential zone 1) where the gross floor area of the dwelling house, including outbuildings, is generally likely to be less than 100 m^2 but more than 55 m^2. This group includes low-cost housing schemes where the gross floor area of the dwelling house, including

outbuildings and allowing for extensions by the owner, would generally not exceed 100 m^2. Restrictions in force control the height and area of buildings, materials used in their construction and the distances from common erf boundaries. Attached dwelling units are separated by a fire wall with a minimum fire-resistance rating of one hour

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- Low-risk – group 4: Residential areas (residential zone 1) where the gross floor area of the dwelling house, including outbuildings, is generally not more than 55 m^2. This group includes low-cost housing schemes. Restrictions in force control the height and area of buildings, materials used in their construction and the distances from common erf boundaries. Attached dwelling units are separated by a fire wall with a minimum fire-resistance rating of one hour.

Fire protection in general

In the case of areas not yet developed, a subdivision of the planned layout into areas or zones according to the relevant fire-risk category should be made, taking into account possible planning parameters such as floor space ratios, height restrictions and building-material restrictions.

When water reticulation systems are being designed for industrial areas, these areas should generally be classified as moderate-risk.

Where the reticulation in an industrial area has been designed on the basis of a moderate-risk classification, a limited number of high fire-risk types of industry can subsequently be permitted to be established in the area without warranting a re-classification of the area to the high-risk category. In this case, the approval conditions for the establishment of the industry should specify the provision of the extra water for fire-fighting (as deemed necessary by the fire department) which is over and above that allowed for in the design of the reticulation. Such provision could take the form of an additional supply to the site, or storage of water on the premises, and would be provided at the cost of the applicant.

Examples of high-risk types of development are:

- timber-storage yards;
- timber-clad buildings;
- institutional buildings and buildings in which hazardous processes are carried out; and
- areas where combustible materials are stored which, because of the quantity of such materials, the extent of the area covered by the materials and the risk of fire spread, may be deemed high-risk.

In the case of existing developed areas, a survey of the fire hazards of the area should be made at intervals of not more than three years. Such a survey should take account of the height, the type of construction, the occupancy of the buildings, the means of approach to buildings, the water supply available and any other feature affecting the fire risk.

Should such a survey indicate the need for re-classification of an area into a higher risk category, then the area should be re-classified accordingly, and the chief fire officer, in conjunction with the local authority engineer, must prepare a report for submission to the controlling authority, requesting that the fire service be upgraded according to the new classification.

Water supply for fire-fighting

The elements in a water reticulation system for the supply of water for fire-fighting are:

- *trunk main:* the pipeline used for bulk water supply;
- *water storage:* reservoir and elevated storage;
- *reticulation mains:* the pipelines in the reticulation to which hydrants are connected; and
- *hydrants:* these may be of the screw-down or sluice-valve type.

The capacity of the above elements is determined according to the category of fire risk applicable.

The fire flow and hydrant flow for which the water reticulation is designed should be available to the fire-fighting team at all times. Close liaison between the water department of the local authority and the fire service should be maintained at all times, so that the water department can be of assistance in times of emergency – for example, isolating sections of the reticulation in order to increase the quantity of water available from the hydrants at the scene of the fire.

Note: *Low-risk – Group 4 category*

No specific provision for fire-fighting water is made in trunk mains, water storage, or reticulation mains in these areas. Hydrants should, however, be located at convenient points in the area on all mains of 75 mm nominal internal diameter and larger, and in the vicinity of all schools, commercial areas and public buildings.

Fire-fighting in areas zoned Low-risk – Group 4 should generally be carried out using trailer-mounted water tanks or fire appliances that carry water, which can if necessary be replenished from the hydrants provided in the reticulation.

Development within areas falling into the Low-risk – Group 4 category, and which qualify for inclusion in higher risk categories, should be protected according to the relevant category as outlined in the Code of Practice and these Guidelines.

Design of trunk mains

The mains supplying fire-risk areas should be designed so that the supply is assured at all times.

Trunk mains serving fire-risk areas should be sized for a design flow equivalent to the sum of the design instantaneous peak domestic demand for the area served by it, and the fire flow given in Table 9.18.

Where an area served by the trunk main incorporates more than one risk category, then the fire flow adopted should be for the highest risk category pertaining to the area.

Water storage

Ground reservoirs

The storage capacity of reservoirs serving fire areas should, over and above the allowance for domestic demand, include for the design fire flow obtained from Table 9.18 for a duration at least equal to that given in Table 9.19.

Where an area served incorporates more than one risk category, then the design fire flow and duration used should be for the highest risk category pertaining to the area served by the reservoir.

Elevated storage

There need be no provision for storage of water for fire-fighting when the pumping plant serving the elevated storage facility is sized to deliver the sum of the instantaneous peak demand and the fire flow obtained from Table 9.18, or the instantaneous peak demand plus an allowance of 20%, whichever is the greater. A standby pumping plant should also be provided.

Reticulation mains

Reticulation mains in fire areas should be designed according to the design domestic demand required. The mains should, however, have sufficient capacity to satisfy the criteria given in Table 9.20.

The minimum residual head should be obtained with the hydrant discharging at the minimum hydrant flow rate, assuming the reticulation is operating under a condition of instantaneous peak domestic demand at the time.

The water reticulation design for each reservoir supply zone should be checked for compliance with this paragraph on the minimum basis of the following:

High-risk and moderate-risk areas

The group of hydrants (see Table 9.18) included in each critical area in the reticulation are in use simultaneously. Critical areas should be checked individually.

Low-risk areas

- <u>Areas of less than 2 000 dwelling units:</u> assume one hydrant in use at a time; each critical area should be checked individually;
- <u>Areas of 2 000 and more dweling units:</u> assume a hydrant in each critical area in use simultaneously on the basis of one hydrant per 2 000 dwelling units or part thereof.

Table 9.19: Duration of design fire flow

FIRE-RISK CATEGORY	DURATION OF DESIGN FIRE FLOW (h)
High-risk	6
Moderate-risk	4
Low-risk – Group 1	2
Low-risk – Group 2	1
Low-risk – Group 3	1
Low-risk – Group 4	N/A

Table 9.18: Design fire flow

FIRE-RISK CATEGORY	MINIMUM DESIGN FIRE FLOW(ℓ/min)	MAXIMUM NUMBER OF HYDRANTS DISCHARGING SIMULTANEOUSLY
High-risk	12 000	All hydrants within a radius of 270 m of the fire
Moderate-risk	6 000	
Low-risk – Group 1	900	1
Low-risk – Group 2	500	1
Low-risk – Group 3	350	1
Low-risk – Group 4	N/A	N/A

Table 9.20: Fire-flow design criteria for reticulation mains

FIRE-RISK CATEGORY	MINIMUM HYDRANT FLOW RATE FOR EACH HYDRANT (ℓ/min)	MINIMUM RESIDUAL HEAD (m)
High-risk	1 500*	15
Moderate-risk	1 500*	15
Low-risk – Group 1	900	7
Low-risk – Group 2	500	6
Low-risk – Group 3	350	6
Low-risk – Group 4	N/A	N/A

*With a design maximum of 1 600 ℓ/min per hydrant

Hydrants

Hydrants should not be provided off mains smaller than 75 mm diameter.

Hydrants should be located in vehicular thoroughfares and opposite erf boundary pegs, and according to Table 9.21.

The hydrants for the high-risk and moderate-risk categories should be the 75 mm diameter sluice-valve type. For the low-risk category, the hydrant may be the screw-down type.

In the case of new developments, hydrant types should be similar to those used by an adjacent metropolitan area, if relevant.

The location of hydrants should be clearly indicated.

Hydrants should be serviced and the flow rate checked for conformity with Table 9.20 at intervals not exceeding one year.

Isolating valves

In reticulation networks, isolating valves should be provided so that not more than four valves need to be closed to isolate a section of main, and so that the total length of main included in an isolated section does not exceed a nominal 600 m.

Valves should be located at street corners opposite erf corner boundary (splay) pegs, and intermediate valves opposite the common boundary peg between two erven.

Fire protection in developing and rural areas

Fire protection in developing and rural areas needs special attention. Designers of water-reticulation systems should accept that the local unavailability of fire-fighting resources at the time of design will most certainly change during the lifetime of the scheme. It is not suggested that the design of systems be according to the guidelines given for developed areas, but designers should consider the following:

- Whatever fire-fighting equipment is provided should be compatible with that of the surrounding area; close liaison with the responsible emergency services agency is considered essential.
- The provision of permanent equipment – like fire hydrants at business sites and institutional erven – should be considered and the type and location should be chosen to minimise vandalism or illegal use.
- The accessibility of the area and the space required for vehicles used by fire-fighting services should be taken into account.

Table 9.21: Location of hydrants

FIRE-RISK CATEGORY	LOCATION OF HYDRANTS
High-risk	distance apart: 120 m maximum
Moderate-risk	distance apart: 180 m maximum
Low-risk – Group 1	distance apart: 240 m maximum
Low-risk – Group 2	distance apart: 240 m maximum
Low-risk – Group 3	distance apart: 240 m maximum
Low-risk – Group 4	Convenient points on mains 75 mm diameter and larger

- Dams and rivers could be utilised by placing hydrants close by for the purpose of pumping water from them.
- Close liaison with the community is required to determine what fire-protection arrangements exist and whether these should be revised.
- Where water-reticulation networks are designed, consideration should be given to their possible future upgrading, and allowance should be made for fire flow in pipelines, taking into consideration the other factors mentioned above. Reference should also be made to "skeletonising" (see Glossary).

GLOSSARY

Balancing storage refers to the volume required in a storage reservoir to cater for peak flows while receiving a constant inlet flow.

Developing areas are considered to be those areas where the services to be installed are subject to future upgrading.

Developed areas are considered to be those areas where the services installed are not subject to future upgrading (i.e. the services are already at the highest intended level).

Fire hose: flexible pipe used for fire-fighting purposes, suitable for connection to a hydrant.

Fire flow: the rate of flow of water required by the fire-fighting service for the extinguishing of fires.

Hydrant: a valve-controlled outlet on the water reticulation system to which a fire hose can be attached, either directly or with an adaptor or standpipe.

Hydrant flow: the discharge rate of water from a hydrant.

Package water treatment plant: a prefabricated purification plant that is assembled on site. It may or may not require small civil construction works and piping for complete functioning.

Peak factor: a dimensionless value, indicating the relationship between peak consumption and average consumption.

Residual head: the pressure in the water main at the hydrant take-off point while the hydrant is discharging.

Skeletonising: is the practice of designing and installing major reticulation pipes for a future higher level of service (which implies that the system will have spare capacity under present conditions.)

Turbidity: a measure of the resistance of water to the passage of light through it; it is caused by suspended or colloidal matter in the water.

Water transmission: means the transport of water from the source to the treatment plant (if there is one) and onward to the distribution area.

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Chapter 10

Sanitation

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SCOPE

The guidelines for sanitation provision cover aspects that need to be considered when planning and implementing sanitation projects for existing residential areas and developing communities. The guidelines will also be of assistance where the Water Services Authority compiles a Water Services Development Plan (the latter forms part of the municipality's Integrated Development Plan).

The guidelines will also assist in determining and setting objectives, developing a strategy and identifying the required planning activities for implementing sanitation projects. Technical guidelines are given for use in feasibility studies and the detailed design of sanitation provision.

The guidelines form part of a planned series of management guidelines intended for use by decision makers. The series of guidelines is shown in Table 9.1 of Chapter 9: Water Supply.

INTRODUCTION

Water services (i.e. water supply and sanitation) in South Africa are controlled by the Water Services Act (Act 108 of 1997) and the National Water Act (Act 36 of 1998). The Water Services Act deals with water services provision to consumers, while the National Water Act deals with water in its natural state.

As in the case of water supply, the provision of sanitation to a community should take place in terms of the relevant Water Services Development Plan, which is required in terms of the Water Services Act. The Water Services Development Plan (which should, of course, be compiled taking cognisance of the National Sanitation Policy) defines the minimum level of sanitation as well as the desired level of sanitation for communities that must be adhered to by a Water Services Provider in its area of jurisdiction. It describes the arrangements for water services provision in an area, both present and future. Water services are also to be provided in accordance with regulations published in terms of the Water Services Act.

The provision of appropriate sanitation to a community should take place in accordance with national policy. Among the major aims set out in the National Sanitation Policy are the following:

- to improve the health and quality of life of the whole population;
- to integrate the development of a community in the provision of sanitation;
- to protect the environment; and
- to place the responsibility for household sanitation provision with the family or household.

The minimum acceptable basic level of sanitation is (Department of Water Affairs & Forestry, 2001):

- appropriate health and hygiene awareness and behaviour;
- a system for disposing of human excreta,household wastewater and refuse, which is acceptable and affordable to the users, safe, hygienic and easily accessible, and which does not have an unacceptable impact on the environment; and
- a toilet facility for each household.

The safe disposal of human excreta alone does not necessarily mean the creation of a healthy environment. Sanitation goes hand in hand with an effective health-care programme. The importance of education programmes should not be overlooked, and the Department of Health is able to assist in this (refer to the following section "Hygiene in sanitation projects").

Sanitation education is part of the National Sanitation Policy and should embrace proper health practices, such as personal hygiene, the need for all family members (including the children) to use the toilet and the necessity of keeping the toilet building clean. Education should also include the proper operation of the system, such as what may and may not be disposed of in the toilet, the amount of water to add if necessary, and what chemicals should or should not be added to the system. The user must also be made aware of what needs to be done if the system fails or what options are available when the pit or vault fills up with sludge.

Current policy is that the basic minimum facility should be a ventilated improved pit (VIP) toilet, or its equivalent. In this chapter, therefore, levels of service lower than this will not be discussed.

The use of ponds is frequently considered for dealing with wastewater in rural areas, and hence their inclusion in this document.

The five main criteria to be considered when providing a sanitation system for a community are:

- reliability;
- acceptability;
- appropriateness;
- affordability; and
- sustainability.

With these criteria in mind, designers should note the following principles from the White Paper on Basic

Household Sanitation, September 2001:

Sanitation improvement must be demand-responsive, and supported by an intensive health and hygiene programme

Household sanitation is first and foremost a household responsibility and must be demand-responsive. Households must recognise the need for adequate toilet facilities for them to make informed decisions about their sanitation options. For users to benefit maximally, they must also understand the link between their own health, good hygiene and toilet facilities.

Community participation

Communities must be fully involved in projects that relate to their health and well-being, and also in decisions relating to community facilities, such as schools and clinics. Communities must participate in decision-making about what should be done and how, contribute to the implementation of the decisions, and share in the benefits of the project or programme. In particular, they need to understand the cost implications of each particular sanitation option.

Integrated planning and development

The health, social, and environmental benefits of improved sanitation are maximised when sanitation is planned for and provided in an integrated way with water supply and other municipal services.

The focal mechanism of achieving integrated planning and development is the municipality-driven Integrated Development Planning (IDP) process (of which the Water Services Development Plan is a component).

Sanitation is about environment and health

Sanitation improvement is more than just the provision of toilets; it is a process of sustained environment and health improvement. Sanitation improvement must be accompanied by environmental, health and hygiene promotional activities.

Basic sanitation is a human right

Government has an obligation to create an enabling environment through which all South Africans can gain access to basic sanitation services.

The provision of access to sanitation services is a local government responsibility

Local government has the constitutional responsibility to provide sanitation services.

Provincial and national government bodies have a constitutional responsibility to support local government in a spirit of co-operative governance.

“Health for all” rather than “all for some”

The use of scarce public funds must be prioritised, in order to assist those who are faced with the greatest risk to health due to inadequate sanitation services.

Equitable regional allocation of development resources

The limited national resources available to support the incremental improvement of sanitation services should be equitably distributed throughout the country, according to population, level of development, and the risk to health of not supporting sanitation improvement.

Water has an economic value

The way in which sanitation services are provided must take into account the growing scarcity of good quality water in South Africa.

The “polluter pays” principle

Polluters must pay the cost of cleaning up the impact of their pollution on the environment.

Sanitation services must be financially sustainable

Sanitation services must be sustainable, in terms of both capital and recurrent costs.

Environmental integrity

The environment must be protected from the potentially negative impacts of sanitation systems.

PLANNING

The various planning stages and other information related to the planning of projects are fully described in Chapter 9 (Water supply). The planning steps and approach with respect to sanitation do not differ from those described in that chapter, and are not repeated here. The reader is thus referred to Chapter 9 for planning information.

HYGIENE IN SANITATION PROJECTS

An introductory discussion on hygiene in water and sanitation projects can be found in Chapter 9.

For people to change their cultural practices and behaviours – some of which are developed around deeply seated values – a lot of motivation is required, accompanied by marketing of the new or modified practices. For instance, when people are accustomed to defecating in the open (free of charge) it will take a lot of effort to motivate them to install toilets (at a cost), and to use those toilets, which come in as a new practice. Furthermore, the motivation for installing toilets or practising a higher level of hygiene must clearly emphasise the benefits to the individual and to the community.

The main components of a hygiene-promotion and -education strategy in a project should include the following:

- motivation and community mobilisation;
- communication and community participation;
- user education (operation and maintenance);
- skills training and knowledge transfer;
- development of messages;
- presentation of messages; and
- maintenance of good practice.

Many of these components overlap, or cut across the whole programme of implementation. For instance, motivation and community mobilisation would need to be maintained throughout the programme and after. The same goes for communication, which is required at all stages of a hygiene-promotion and -education programme. In practice there is no particular cut-off point between one component and another.

The following should be implemented during the projects' stages (see Appendix A) to ensure that water supply and sanitation projects have a positive impact on the quality of life and level of hygiene in communities:

- *Development and structuring of a strategic plan for the implementation of hygiene promotion and education in water and sanitation projects*: This strategic plan should fit in and dovetail with national and provincial strategies for health and hygiene in South Africa, and should include advocacy, training and capacity-building, implementation, monitoring and evaluation.

- *Liaison with other programmes/projects active in the health and hygiene field*: Other initiatives regarding hygiene (PHAST, etc) should be identified and coordinated to prevent duplication, and to optimally address the needs of government and the communities. A number of other activities and programmes in schools, clinics, hospitals and the media (TV, radio, newspapers, magazines), should be identified and coordinated for a broader impact of hygiene promotion and education.

- *Informing and training local government structures, environmental health officers (EHOs), non-governmental organisations (NGOs), community-based organisations and consultants*: A programme should be developed to inform and train all local, regional and national institutions and structures involved in water supply and sanitation, health and hygiene promotion and education.

- *Monitoring and evaluating the implementation of hygiene in water and sanitation projects*: A body or organisation should be established and tasked to monitor the quality and standard of hygiene promotion and education. The results of hygiene promotion and education should be evaluated against key health indicators set up at the start of a project.

- *Disseminating information regarding hygiene in water and sanitation projects*: The process and results of hygiene promotion and education in a project should be disseminated in articles, conference papers, reports, seminars and workshops with other researchers in the field of health and hygiene.

TECHNICAL INFORMATION

Before a sanitation system is selected, the available options should be examined. This section describes the various systems and factors that could influence the selection. Detailed design information is not included in this chapter. Instead, reference is made to various publications in which all the required information may be found. Some general background information on the different sanitation technologies is, however, included.

Only sanitation systems commonly used or accepted in South Africa are described.

Categories of sanitation systems

There are two ways to handle human waste. It can either be treated on site before disposal, or removed from the site and treated elsewhere. In either case, the waste may be mixed with water or it may not. On this basis the following four groups may be distinguished:

Group 1: No water added – requiring conveyance.

Group 2: No water added – no conveyance.

Group 3: Water added – requiring conveyance.

Group 4: Water added – no conveyance.

Table 10.1: Categories of sanitation systems

	REQUIRING CONVEYANCE (off-site treatment)	NO CONVEYANCE REQUIRED (treatment, or partial treatment, on site; accumulated sludge also requires periodic removal)
NO WATER ADDED	GROUP 1 Chemical toilet (temporary use only).	GROUP 2 Ventilated improved pit toilet. Ventilated improved double-pit toilet. Ventilated vault toilet. Urine-diversion toilet.
WATER ADDED	GROUP 3 Full waterborne sanitation. Flushing toilet with conservancy tank. Shallow sewers.	GROUP 4 Flushing toilet with septic tank and subsurface soil absorption field. Low-flow on-site sanitation systems (LOFLOs): Aqua-privy toilet.

Table 10.1 illustrates the sanitation systems associated with each of the above groups. Note that some of the systems fall somewhere between the four categories as, for example, where solids are retained on each property and liquids are conveyed from site, or where water may be added, but only in small quantities. Since increasing the number of categories would complicate the table unnecessarily, these systems have been included in the categories that best describe the treatment of the waste. The operating costs of systems in which waste is conveyed and treated elsewhere can be so high that these systems may in the long term be the most expensive of all. The capital and installation costs of any conveyance network using large quantities of clean water to convey small quantities of waste are very high, and a possibly inappropriately high level of training and expertise is also required to construct and maintain such systems.

Developers should consider all the sanitation alternatives available before deciding on the most appropriate solution for the community in question. A solution that may be appropriate in one community may be a total failure in another because of cost, customs and religious beliefs, or other factors. A solution must also not be seen as correct purely because developers and authorities have traditionally implemented it.

Description of sanitation systems

The main types of sanitation systems are described below. The list is not complete and many commercial manufacturers provide systems that may be a variant of one or more.

The advantages and disadvantages of each system will depend on its particular application. What may be a disadvantage in one situation may in fact be an advantage in another. Thus the advantages and disadvantages listed after each description should be seen not as absolutes, but merely as aids to selecting the right sanitation system for a particular application.

Group 1: No water added – requiring conveyance

(for treatment at a central treatment works)

Chemical toilets (not a preferred option)
A chemical toilet stores excreta in a holding tank that contains a chemical mixture to prevent odours caused by bacterial action. The contents of the holding tank must be emptied periodically and conveyed to a sewage works for treatment and disposal. Some units have a flushing mechanism using some of the liquid in the holding tank to rinse the bowl after use. The chemical mixture usually contains a powerful perfume as well as a blue dye. Chemical toilets can range in size from the very small portable units used by campers to the larger units supplied with a hut. The system can provide an instant solution and is particularly useful for sports events, construction sites or other temporary applications where the users are accustomed to the level of service provided by a waterborne sanitation system. The system can also be used where emergency sanitation for refugees is required, in which case it can give the planners the necessary breathing space to decide on the best permanent solution. It should not be considered as a permanent sanitation option.

Factors to consider before choosing this option are the following:

- Chemical toilets have relatively high capital and maintenance costs.
- It is necessary to add the correct quantity of chemicals to the holding tank.

- Periodic emptying of the holding tank is essential; this requires vacuum tankers, so access should be possible at all times.
- The system only disposes of human excreta and cannot be used to dispose of other liquid waste.
- The units can be installed very quickly and easily.
- They can be moved from site to site.
- They require no water connection and require very little water for operation.
- They are hygienic and free from flies and odour, provided that they are operated and maintained correctly.
- The chemicals could have a negative effect on the performance of wastewater treatment works.

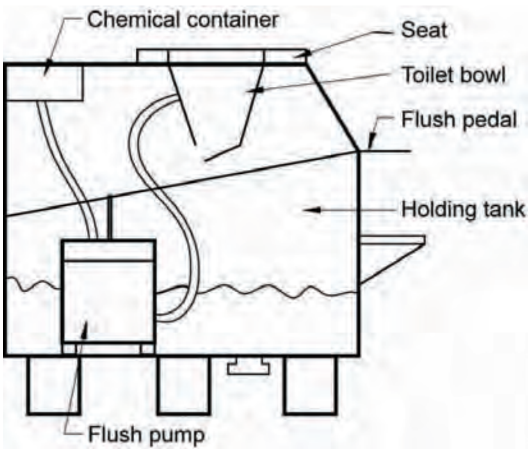

Figure 10.1: Chemical toilet unit

Group 2: No water added – no conveyance
(treatment or partial treatment on site before disposal)

Ventilated improved pit (VIP) toilet
The VIP toilet is a pit toilet with an external ventilation pipe. It is both hygienic and relatively inexpensive, provided that it is properly designed, constructed, used and maintained. Detailed information on VIP design is available in the publication *Building VIPs* by Bester and Austin (1997), which is obtainable from the Department of Water Affairs & Forestry, Pretoria. Information on different soil conditions, as they affect the building of VIP toilets, is also included. A SABS Code of Practice for the construction of VIPs is currently under preparation by the SABS. There are also variations of VIP toilets available – the Archloo, Phungalutho, Sanplat, etc. Those mentioned have been used with success and are firmly established in the industry.

It is possible to construct the entire toilet from local materials, although it is more usual to use commercial products for the vent pipe and the pedestal. Several toilet superstructures are also commercially available. When the pit is full, the superstructure, pedestal, vent pipe and slab are normally moved to a freshly dug pit and the old pit is covered with soil. The VIP toilet can be made more permanent by lining the pit with open-jointed brickwork or other porous lining. The pit can then be emptied when required, using a suitable vacuum tanker, without the danger of the sides of the pit collapsing. Water (sullage) poured into the pit may increase the fill-up rate (depending on soil conditions) and should be avoided; this sanitation technology is therefore not recommended where a water supply is available on the site itself.

Factors to consider before choosing this option:

- If the stand is small there may be insufficient space to allow continual relocation of the toilet; therefore arrangements should exist for emptying the toilet.
- Unlined pit walls may collapse.
- The excreta are visible to the user.
- The system may be unable to drain all the liquid waste if large quantities of wastewater are poured into the pit.
- The cost of the toilet is relatively low; it provides one of the cheapest forms of sanitation while maintaining acceptable health standards.
- The toilet can be constructed by the recipients, even if they are unskilled, as very little training is required.
- Locally available materials can be used.
- If required, the components can be manufactured commercially and erected on a large number of plots within a short space of time.
- The system is hygienic, provided that it is used and maintained correctly.
- The system can be used in high-density areas only if a pit-emptying service exists.

- The system can be upgraded at a later stage to increase user convenience (e.g. to a urine-diversion toilet).
- The system cannot ordinarily be installed inside a house.
- The quantity of water supplied to the site should be limited.
- If a pit-emptying service exists, proper access to the pit should be provided.

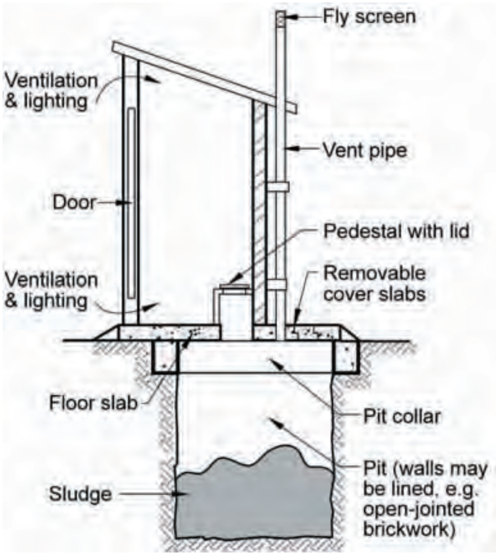

Figure 10.2: VIP toilet

<u>Ventilated improved double-pit (VIDP) toilet</u>
The VIDP toilet, also known as the twin-pit composting toilet, was developed mainly for use in urban areas where, because of limited space on the smaller plots, it may be impossible to relocate the toilet every time the pit becomes full. The VIDP is a relatively low-cost and simple but permanent sanitation solution for high-density areas. Two lined shallow pits, designed to be emptied, are excavated side by side and are straddled by a single permanent superstructure. The pits are used alternately: when the first pit is full it is closed and the prefabricated pedestal is placed over the second pit. After a period of at least one year the closed pit can be emptied, either manually (if this is culturally acceptable) or mechanically, and then it becomes available for re-use when the other pit is full. Each pit should be sized to last a family two to three years before filling up. It is important that the dividing wall between the pits be sealed, to prevent liquids seeping from the pit currently in use into the closed pit, thus contaminating it. The VIDP toilet can be built partially above the ground in areas where there is a high water table, but the distance between the pit floor and the highest water table should be at least 1 m.

Detailed information on VIDP design is also available in the publication *Building VIPs* by Bester and Austin (1997), obtainable from the Department of Water Affairs & Forestry, Pretoria.

Factors to consider before choosing this option are similar to those for the VIP, but include the following:

- Some training is required to ensure that the pit lining is properly constructed.
- The contents of the used pit may be safely used as compost after a period of about two years in the closed pit.
- The user may not be prepared to empty the pit, even though the contents have composted.
- The superstructure can be a permanent installation.
- The system can be regarded as a permanent sanitation solution.
- The system can be used in high-density areas.
- The system can be used in areas with hard ground, where digging a deep pit is impractical.

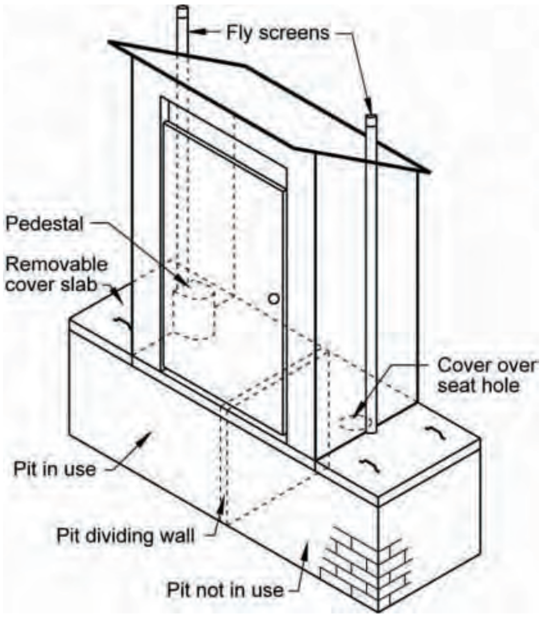

Figure 10.3: VIDP toilet

<u>Ventilated vault (VV) toilets</u>
The VV toilet is basically a VIP toilet with a watertight pit that prevents seepage. It can be regarded as a low-cost, permanent sanitation solution, especially in areas with a high groundwater table or a poor capacity for soil infiltration, or where the consequences of possible groundwater pollution are unacceptable.

The amount of wastewater disposed of into the vault should be limited to avoid the need for frequent emptying, so it is advisable not to use this type of sanitation technology where a water supply is available on site. Should an individual water connection be provided to each stand at a later date, as part of an upgrading scheme for the residential area, then the vault can be utilised as a solids-retention tank (digester) and liquids can be drained from the site using a settled-sewage system, also called a solids-free sewer system (see the section on "settled-sewage systems" under Group 3). Ventilation, odour and insect control operate on the same basis as the VIP toilet.

Factors to consider before choosing this option are similar to those of the VIP, but include the following:

- It is necessary to make use of a vacuum-tanker service for periodic emptying of the vaults.
- Builder training and special materials are required if a completely waterproof vault is required.
- The system can be used in areas with a high water table if the vault is properly constructed.
- The system can be used in areas where pollution of the groundwater is likely if a VIP toilet system is used.
- The system can be used in high-density areas.
- This system provides very good opportunities for upgrading since the vault can be used as a solids-retention tank when upgrading to solids-free sewers.

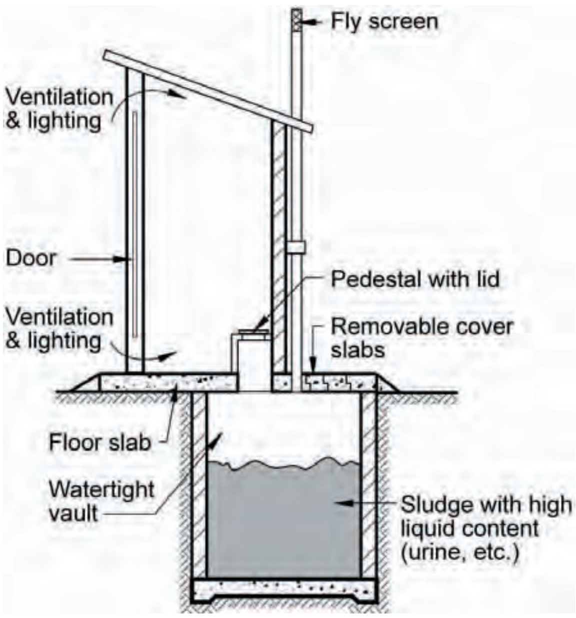

Figure 10.4: Ventilated vault toilet

- The cost of emptying equipment and the operation of a vacuum-tanker service could be excessive (see also the section on vacuum tankers).

<u>Urine-diversion (UD) toilet</u>

The urine-diversion toilet, also known as the "dry-box", is a superior type of dry toilet that circumvents the problems sometimes encountered with the implementation of VIP toilets, namely unfavourable geotechnical or hydrological conditions, high-density settlements with small erven, etc. The main advantage is that a pit is not required, so the toilet may be installed inside the house, if desired by the owner. Urine is diverted at source by a specially designed pedestal and, owing to the relatively small volumes involved, may simply be led into a shallow soakpit. Alternatively, urine can be easily collected in a container and re-used for agricultural fertiliser, as it is rich in plant nutrients such as nitrogen, phosphorus and potassium. Faeces are deposited in a shallow vault and covered with a sprinkling of ash or dry soil, which absorbs most of the moisture. They are further subjected to a dehydration process inside the vault, which hastens pathogen die-off. Depending on the temperature and degree of desiccation attained in the vault, the faeces may be safe to handle after a period of six to eighteen months, and can then be easily removed from the vault and either disposed of or re-used as soil conditioner, depending on individual preferences.

Other favourable aspects of this type of toilet are an absence of odours or flies (if it is properly used), a relatively low capital cost that may, depending on the specific circumstances, be even less than a VIP toilet, and a negligible operating cost. There are also environmental advantages, due to the fact that no pit is required.

Detailed guidelines on the implementation of this technology are contained in the publication *Urine-diversion ecological sanitation systems in South Africa* by Austin and Duncker (CSIR, 2002).

Factors to consider before choosing this option:

- The technology is well suited to dense urban settlements and places where environmental conditions do not favour other types of sanitation.
- Use of the toilet requires an adherence to certain operational requirements, and a proper commitment from owners is required. Good user education is therefore especially important.
- Building materials similar to those for a VIP toilet may be used, and the toilets can be

constructed by relatively unskilled persons. Components can also be manufactured commercially.

- The system is hygienic, provided that it is used and maintained correctly. Safe re-use of the urine and faeces is also facilitated.
- The system can be installed inside a house, if desired.
- There may be reluctance on the part of the user to empty the vault, even though the contents are innocuous.
- The system can be regarded as a permanent sanitation solution that will never need upgrading.

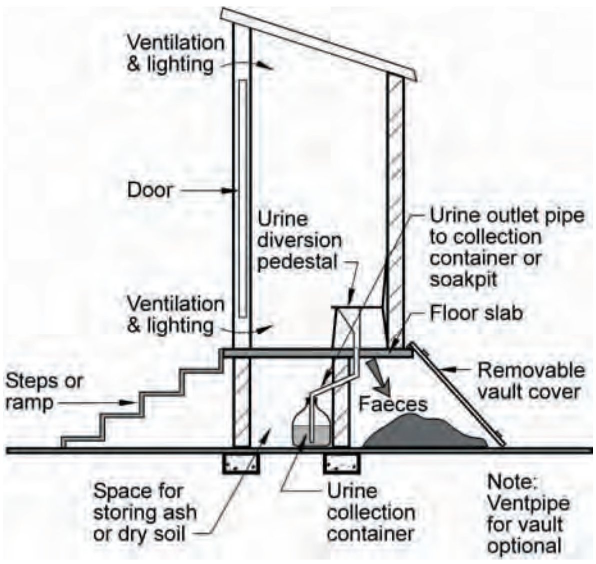

Figure 10.5: Urine-diversion toilet

Group 3: Water added – requiring conveyance
(treatment at a central works)

<u>Full waterborne sanitation</u>
This is an expensive option and requires ongoing maintenance of the toilet installation, the sewer reticulation and the treatment works, and the recipient community should be informed accordingly. The system requires a water supply connection to each property. The water is used to flush the excreta from the toilet pan and into the sewer, as well as to maintain a water seal in the pan. The excreta are conveyed by the water, in underground pipes, to a treatment works that may be a considerable distance from the source. The treatment works must be able to handle the high volume of liquid required to convey the excreta. The quantity of water required (usually 6-10 litres per flush) can be reduced by using low-flush pans designed to flush efficiently with as little as three litres. Research has indicated that the operation of the sewer system is not adversely affected by low-volume flush toilets. Flush volumes of 8-9 litres are normally used, however. In an area where water is costly or scarce, it may be counter-productive to purify water only to pollute it by conveying excreta to a treatment and disposal facility.

Factors to consider before choosing this option are the following:

- The system is expensive to install, operate and maintain.
- The system can be designed and installed only by trained professionals.
- The treatment works must be operated and maintained properly if pollution of waterways is to be avoided.
- The system requires large amounts of water to operate effectively and reliably.
- The system is hygienic and free of flies and odours, provided that it is properly operated and maintained.
- A high level of user convenience is obtained.
- The system should be regarded as a permanent sanitation system.
- The toilet can be placed indoors.
- This system can be used in high-density areas.
- An adequate, uninterrupted supply of water must be available.

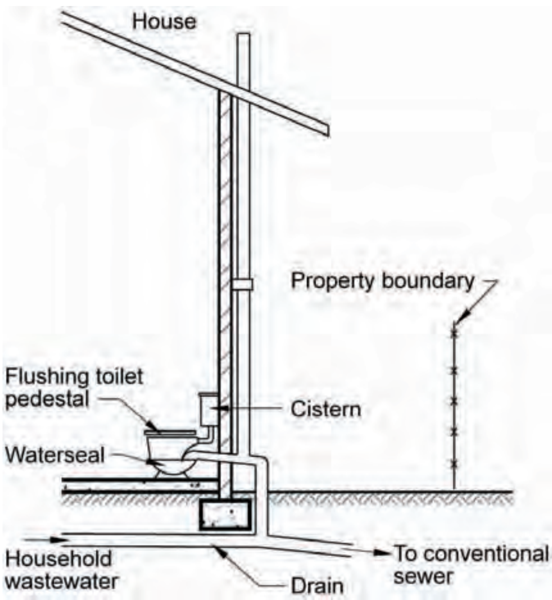

Figure 10.6: Waterborne sewerage system

Shallow sewers

The shallow sewer system is basically a conventional system where a more simple approach with respect to design and construction is followed. Basically it entails a system where gradients are flatter, pipes are smaller and laid shallower, manholes are smaller and constructed of brickwork, and house connections are simpler. Where such systems are installed, community involvement in management and maintenance issues is preferred.

Design of sewer networks:

The professional responsibility in design remains with the engineer, and guidelines should never be regarded as prescriptive. Most local authorities have their own requirements and preferences on technical detail, such as pipe slopes, manhole details, materials, and so on. These are based on their own particular experiences and new designs should therefore be discussed with the relevant controlling authority.

Designers should not assume that sewer systems will always be properly maintained, and allowance should be made for this.

The hydraulic design of the sewers should be done according to acceptable minimum and maximum velocities in the pipeline. A number of pipe manufacturers have design charts available in their product manuals and these can be used in the absence of other guidelines.

The construction of a system should be in accordance with the relevant sections of SABS 1200:1996. The depth of the sewer is normally determined by its position on site. Sewers in mid-block positions and on sidewalks can normally be laid at shallower depths. Should laying of sewer and water pipes in the same trench be considered, workmanship should be of a high standard. Appendix C gives design guidelines that should be useful.

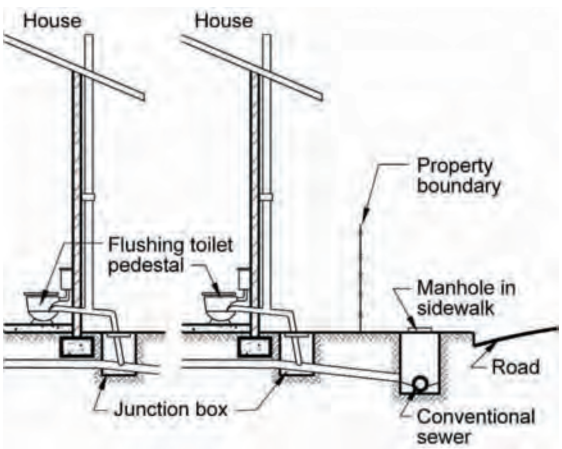

Figure 10.7: Shallow sewer system

Flushing toilet with conservancy tank

This system consists of a standard flushing toilet that drains into a storage or conservancy tank on the property; alternatively, several properties' toilets can drain into one large tank. A vacuum tanker regularly conveys the excrement to a central sewage treatment works for purification before the treated effluent is discharged into a watercourse. The appropriate volume of the conservancy tank should be calculated on the basis of the planned emptying cycle and the estimated quantity of wastes generated. Tank volumes are sometimes prescribed by the service provider.

Factors to consider before choosing this option are the following:

- The system is expensive to install, operate and maintain, although the capital cost is lower than the fully reticulated system.
- The system can be designed and installed only by trained professionals.
- A treatment works must be operated and maintained properly to avoid the pollution of waterways.
- A fleet of vacuum tankers must be maintained by the local authority.
- Regular collection is essential.
- The system is hygienic and free of odours, provided that it is properly operated and maintained.
- A high level of user convenience is obtained.
- The toilet can be placed indoors.
- The system can be used in high-density areas.

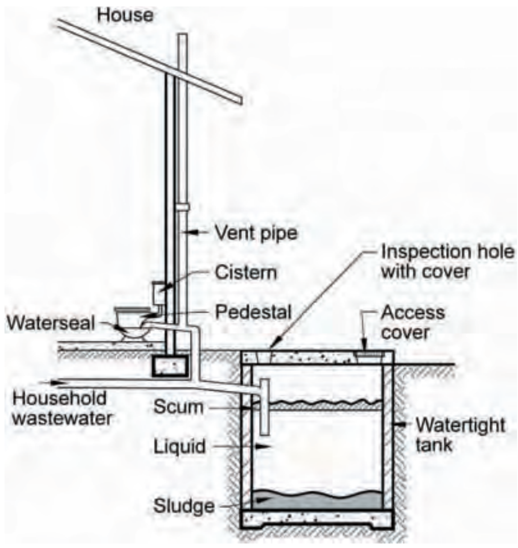

Figure 10.8: Flushing toilet with conservancy tank

- This system has good potential for upgrading since the conservancy tank can be used as a digester when upgrading to a settled-sewage system.

- An adequate, uninterrupted supply of water must be available.

Settled sewage system

In settled sewage systems, also known as solids-free systems or Septic Tank Effluent Drainage (STED), the solid portion of excreta (grit, grease and organic solids) is retained on site in an interceptor tank (septic tank), while the liquid portion of the waste is drained from the site in a small-diameter sewer. Such sewers do not carry solids, and have very few manholes. Tolerances for excavation and pipe laying may be greater than for conventional sewers, allowing lesser skilled labour to be used. Although the liquid portion of the waste must be treated in a sewage works, the biological design capacity of the works can be greatly reduced because partial treatment of the sewage will take place in the retention tank on the site. The tank will also result in a much lower peak factor in the design of both the reticulation and the treatment works. The retention tank should be inspected regularly and emptied periodically, to prevent sludge overflowing from the tank and entering the sewer. This system is an easy upgrading route from septic tanks with soakaways, conservancy tanks, and other on-site systems, as they can be connected to a settled-sewage system with very little modification. Tipping-tray pedestals and water-saving devices can also be used in settled-sewage schemes without fear of causing blockage resulting from the reduced quantity of water flowing in the sewers.

Factors to consider before choosing this option are the following:

- The system requires a fairly large capital outlay if new interceptor tanks have to be constructed (i.e. if there are no existing septic tanks which can be used).

- Care must be taken to ensure that only liquid waste enters the sewers; this may mean regular inspection of the on-site retention tanks.

- Vacuum tankers must be maintained by the local authority.

- The system is hygienic and free of flies and odours, provided that it is properly operated and maintained.

- A high level of user convenience is obtained.

- The system can be regarded as a permanent sanitation system.

- The toilet can be placed indoors.

- All household liquid waste can be disposed of via this sanitation system.

- The sewers can be installed at flatter gradients and can even be designed to flow under pressure.

- The system provides an easy and reasonably priced option when areas with on-site sanitation need to be upgraded, because of increased water consumption and higher living standards.

- The system can be used in high-density areas.

- An adequate uninterrupted supply of water must be available.

- Regular inspection of the septic tank/digester is required to prevent an overflow of sludge.

Technical information on the design of these systems may be found in the CSIR, Division of Building and Construction Technology publication *Septic tank effluent drainage systems* (1997).

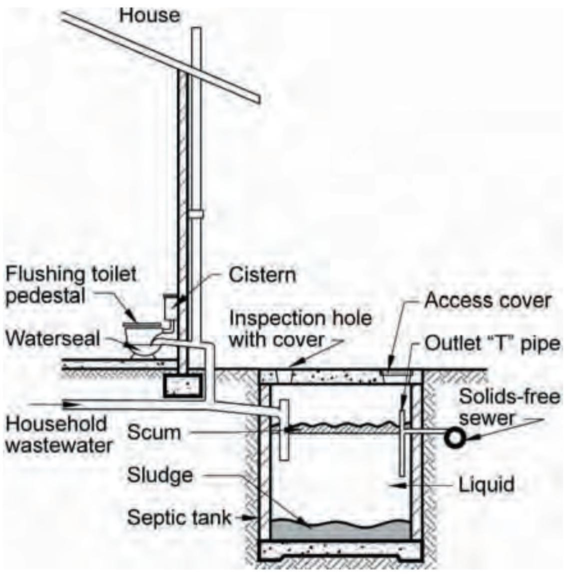

Figure 10.9: Settled sewage system

Group 4: Water added – no conveyance

(treatment or partial treatment on site before disposal)

These systems generally dispose of all or part of the effluent on site. Some systems retain only the solid portion of the waste on site and the liquids are conveyed to a suitable treatment and disposal facility.

Systems that dispose of the liquid fraction on site require a soil percolation system, and the amount of liquid that can be disposed of will depend on the system's design and the permeability of the soil.

Flushing toilet with septic tank and soakaway

Water is used to flush the waste from a conventional toilet pan into an underground septic tank, which can be placed a considerable distance from the toilet. The septic tank receives the sewage (toilet water and sullage (greywater)) and the solids digest and settle to the bottom of the tank in the same manner as in the settled-sewage system. The septic tank therefore provides for storage of sludge. The effluent from the tank can contain pathogenic organisms and must therefore be drained on the site in a subsoil drainage system. The scum and the sludge must be prevented from leaving the septic tank as they could cause permanent damage to the percolation system. It is therefore advisable to inspect the tank at intervals to ascertain the scum and sludge levels. Most tanks can be emptied by a conventional vacuum tanker. Liquid waste from the kitchen and bathroom can also be drained to the septic tank. Septic tanks should be regarded as providing a high level of service.

Factors to consider before choosing this option are the following:

- It is relatively expensive and requires a water connection on each stand.
- It requires regular inspection, and sludge removal every few years, depending on the design capacity of the septic tank.
- Percolation systems may not be suitable in areas of low soil permeability or high residential density.
- It provides a level of service virtually equivalent to waterborne sanitation.
- The system is hygienic and free of flies.
- The toilet can be inside the dwelling.
- This system has excellent potential for upgrading, since the septic tank outlet can easily be connected to a settled-sewage system at a future date.
- An adequate, uninterrupted supply of water must be available.

Designers are referred to the publication *Septic tank systems* (BOU/R9603), available from the CSIR, Division of Building and Construction Technology, for information on the design of septic tanks. Reference to the percolation capacity of soils is made later in this chapter under the section "Evaluation of sites".

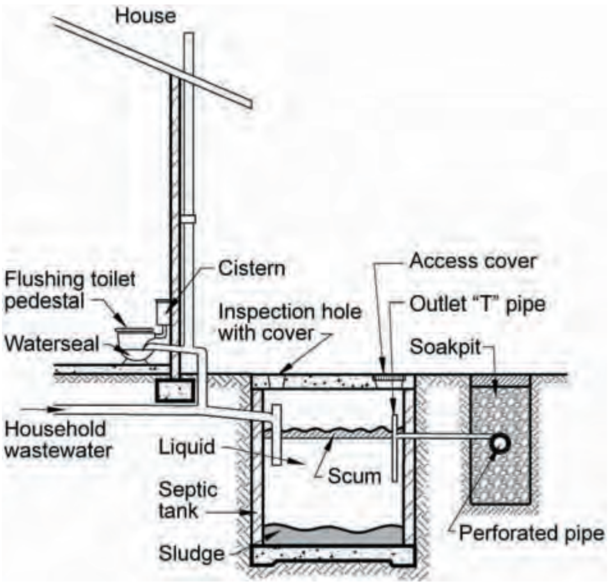

Figure 10.10: Flushing toilet with septic tank and soakaway

Low-flow on-site sanitation systems (LOFLOs)

The term LOFLOs refers to the group of on-site sanitation systems that use low volumes of water for flushing (less than 2,5 litres per flush). These systems include a pedestal, digestion capacity and soakaway component. They are:

- aqua-privies;
- pour-flush toilets* and low-flush systems*; and
- low-flow septic tanks*.

*These types are not generally found in South Africa.

Aqua-privies:

An aqua-privy is a small, single-compartment septic tank directly under or slightly offset from the pedestal. The excreta drops directly into the tank through a chute, which extends 100 mm to 150 mm below the surface of the water in the tank. This provides a water seal, which must be maintained at all times to prevent odour and keep insects away. The tank must be completely watertight; it may therefore be practical to use a prefabricated tank. The tank must be topped up from time to time with water to compensate for evaporation losses if flushing water is not available. This can be done by mounting a wash trough on the outside wall of the superstructure and draining the used water into the tank. The overflow from the tank may contain pathogenic organisms and should therefore run into a soil percolation system (it can also be connected to a settled-sewage system at a later stage).

Factors to consider before choosing this option are the following:

- The excreta are visible to the user.
- The tanks must be completely watertight.
- The user must top up the water level in the aqua-privy to compensate for evaporation losses.
- The system is hygienic, provided that it is used and maintained correctly.
- It is relatively inexpensive.
- The system can be regarded as a permanent sanitation solution.
- This system has excellent potential for upgrading, since the tanks work in the same way as a septic tank and can thus be connected directly to a settled-sewage system.
- The tanks need regular inspection, and sludge removal is required from time to time.
- An adequate, uninterrupted supply of water must be available.

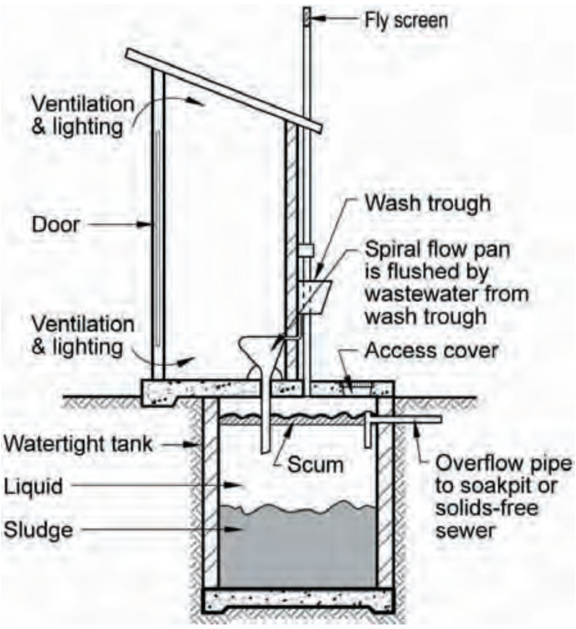

Figure10.11: Aqua-privy toilet

FACTORS AFFECTING THE CHOICE OF A SANITATION SYSTEM

General considerations

The primary reason for installing a sanitation system in a community is to assist in the maintenance of health and should be seen as only one aspect of a total health programme. The choice of a sanitation system by a community will be influenced by several factors, such as the following:

- The system should not be beyond the technological ability of the community insofar as operation and maintenance are concerned.
- The system should not be beyond the community's ability to meet the capital as well as the maintenance costs.
- The system should take into account the level of water supply provided, and possible problems with sullage (greywater) management.
- The likelihood of future upgrading should be considered, particularly the level of service of the water-supply system.
- The system should operate well despite misuse by inexperienced users.
- In a developing area the system should require as little maintenance as possible.
- The system chosen should take into account the training that can be given to the community, from an operating and maintenance point of view.
- The system should be appropriate for the soil conditions.
- The community should be involved to the fullest extent possible in the choice of an appropriate system.
- To foster a spirit of real involvement and ownership, the community should be trained to do as much as possible of the development work themselves.
- Local customs should be carefully considered.
- The local authority should have the institutional structure necessary for the operation and maintenance of the system.
- The existing housing layout, if there is one, should not make the chosen system difficult to construct, maintain or operate.

- Environmental factors should be considered: surface pollution, possible groundwater contamination, etc.

The cost of the system

Many people in developing areas are not only unable to afford sophisticated sanitation systems, but these systems may also be technically inappropriate for them. At the same time, the sanitation alternative with the lowest overall cost may also be inappropriate because of the community's cultural background or because of its unwillingness or inability to operate the system correctly.

When the costs of different systems are compared, all relevant factors should be taken into account. Examples of costs often ignored are the following:

- A pit toilet may require relocation on the site or emptying every 4-10 years, depending on its capacity.

- Sludge from septic tanks and other on-site sanitation systems may require treatment before disposal.

- Training may be required for operators and maintenance staff.

- The community may have to be trained in the use of the system for it to operate effectively.

- Regional installations such as treatment works may be required.

- Special vehicles and equipment may be required for operation or maintenance.

To keep costs to a minimum, several issues are relevant:

- Who pays what? For example, if a government institution or development agency is paying all of the capital costs, then the community will generally demand the most expensive, highest level of sanitation. If, on the other hand, the capital costs are to be recovered from the community, then its choice of sanitation system may be quite different. The lack of income to pay for maintenance could have serious financial implications as well as health risks.

- Would the community prefer lower capital costs and higher maintenance costs, or vice versa?

- Will the cost comparison between options change if all the potential benefits and costs are included?

- Are any of the costs incorrectly or dishonestly represented? For example, have capital grants or soft loans been ignored? Do certain services have hidden subsidies that produce misleading comparisons (for example, where treatment costs are paid by regional authorities)?

Where finance is limited, developers should consult the community, determine its priorities, and seek ways to achieve the improvements desired. This may take extra time but a motivated community will contribute more to successful project implementation and, perhaps more importantly, to the long-term operation and maintenance of the system selected.

Sanitation at public facilities

Sanitation facilities are required at public buildings such as schools and clinics. The large number of people using a concentrated facility can cause problems if there is inadequate on-site drainage and a lack of general maintenance, such as cleaning of the toilet and replacement of toilet paper. Most types of on-site sanitation systems can be used, provided that developers take note of the special requirements for public facilities.

Generally speaking, the system should use as little water and require as little routine maintenance as possible. Before choosing a system that requires daily maintenance for effective operation, one must ensure that maintenance tasks will in fact be performed. A rural school may not be able to afford the services of a janitor to look after routine maintenance.

The number of sanitation units required at schools is covered in the National Building Regulations (SABS 0400:1990). Because of the number of users, care should be taken to prevent pollution of the groundwater, particularly if there is a borehole supplying the school with water.

Urinals that do not require water for flushing are available from commercial sources. These can be used effectively in sanitation facilities. However, they require the weekly addition of a special oil to the trap and the application of a special deodorised cleanser to the bowl or slab. They perform satisfactorily as long as the weekly maintenance is carried out and should therefore be used only where the necessary training can be given to the cleaners and where the supply of oil and cleanser can be assured.

DISPOSAL OF SLUDGE FROM ON-SITE SANITATION SYSTEMS

All forms of on-site sanitation will result in an accumulation of sludge that, at intervals, must be removed from the pit or tank and conveyed to some treatment or disposal facility. If the pit or tank contains fresh sewage, the sludge must be treated or disposed of in a way that will not be harmful to the environment or a threat to health; if the waste matter has been allowed to decompose to the extent where there are no longer any pathogens present, the sludge can be spread on the land as compost.

Sludge can be disposed of only in accordance with the prescribed methods. See Water Research Commission (1997) *Permissable utilisation and disposal of sewage sludge.*

An effective refuse-collection system should be in operation in high-density residential areas and people should be encouraged to use it. If this is not done, the residents will probably use the toilet to dispose of tins, bottles, plastic and other forms of refuse which will result in the pits filling very rapidly. This will present problems when emptying pits with a vacuum tanker. Where regular emptying will be required, it is advisable to construct permanent pits with lined walls to prevent damage during emptying, which could lead to the collapse of the pit walls. Note, however, that pit linings should make allowance for percolation of effluent into the surrounding soil (e.g. by leaving the vertical joints of a brick lining unfilled). A regular programme for pit emptying in an area is better than responding to individual ad hoc calls for pit emptying.

Composition of pit or vault contents

In a sealed tank or vault, human excreta will usually separate into three distinct layers, namely a layer of floating scum, a liquid layer and a layer of sediment. Well-drained pits may have no distinct liquid layer, and therefore no floating scum layer. The scum layer seems to be caused by the presence of paper, oil and grease in the tank and it is also more prominent in tanks with a large number of users. It is usually possible to break up the scum layer without much effort. The water content of pits can vary between 50% and 97%, depending on the type of sanitation system, the personal habits of the users, the permeability of the soil, and the height of the groundwater table. Cognisance should be taken of the fact that different materials used for anal cleansing will have different breakdown periods. Newspaper will require more time to break down than ordinary toilet paper, and in some cases will not break down at all. This will obviously cause the pit to fill up more quickly.

Methods of emptying pits

It is possible to empty pits manually, using scoops and buckets, and to dig out the thicker sludge with spades, but this poses obvious health risks to the workers involved. The use of ventilated improved double-pit toilets overcomes this unpleasantness by allowing the excreta to decompose into a pathogen-free, humus-rich soil, after storage in the sealed pit for about two years. However, many people do not like to empty their pits manually.

The most suitable method of emptying a pit mechanically involves the use of a vacuum tanker, where a partial vacuum is created inside a tank and atmospheric pressure is used to force the pit contents along a hose and into the tank. The use of a vacuum is preferred to other pumping methods, because the contents do not come into contact with the moving parts of the pump, where they can cause damage or blockages. Various techniques can be used to convey this sludge along the pipe to the tank. Thin sludge with a low viscosity can be conveyed by immersing the nozzle below the surface of the sludge, drawing a constant flow into the tank.

Thicker, more viscous sludge requires a pneumatic conveying technique, where the particles of waste matter are entrained in an air stream. This can be achieved by holding the end of the nozzle a few centimetres above the surface of the waste and relying on the high velocity of the air to entrain particles and convey them along the pipe to the holding tank. This technique requires very high-capacity air pumps to operate effectively. Devices such as an air-bleed nozzle can be used to obtain the same effect with less operator skill and smaller air pumps. Pneumatic conveyance can also be achieved with a low-capacity air pump by using the plug-drag (or suck-and-gulp) technique. This method relies on submerging the hose inlet, allowing a vacuum pump to create a vacuum inside the tank, then drawing the hose out to allow a high-velocity air stream to convey a plug of waste into the tank. The plug-drag technique works best with a vehicle with a fairly high-capacity pump (say 10 m^3/min) and a relatively small holding tank (1,5-2 m^3).

Sludge flow properties

Sludge generally exhibits a yield stress, shear thinning behaviour and thixotropy. In effect, this means that the hardest part of the operation is to get the sludge moving. The addition of small quantities of water to the sludge can assist greatly in getting it to move by causing shearing to take place. Being thixotropic, the sludge "remembers" this and exhibits a lower shearing stress next time.

Vacuum tankers

Most vacuum tankers available today are designed for use in developed countries, where roads are good and maintenance is properly done. However, some manufacturers are realising the special conditions that exist in developing countries and are now adapting their designs or, even better, are developing completely new vehicles designed to cope with the actual conditions under which these vehicles will have to work.

The size of conventional vacuum tanker vehicles prevents their gaining easy access to toilets. They are high off the ground, which may limit the depth of pit that can be emptied, and they are so heavy when loaded that travelling over bad or non-existent roads is extremely difficult. The equipment on the vehicle is often complex and requires regular maintenance, which is seldom carried out by the unskilled people who, in most cases, will operate the vehicles. Watery sludge from conservancy tanks and septic tanks may not be a problem, but the tankers often cannot cope with the more viscous sludge found in pit toilets.

A purpose-designed pit-toilet-emptying vehicle should have the following attributes:

- a low mass;
- a low overall height;
- high manoeuvrability;
- ability to travel on extremely poor roads;
- a small-capacity holding tank with a relatively high-capacity pump, to facilitate the use of plug-drag techniques;
- the ability to transfer its load to another vehicle or trailer if there are long haul distances to the disposal site;
- both vehicle and equipment must be robust;
- little skill should be required to operate the vehicle and the equipment;
- capital and operating costs must be as low as possible, to facilitate operation by emerging contractors or entrepreneurs;
- a small pressure pump and water for washing down must be provided; and
- storage compartments must be provided for the crew's personal effects.

Disposal of sludge

Pit-toilet sludge can be disposed of by burial in trenches.

Dehydrated faecal matter from urine-diversion toilets may be safely re-used as soil conditioner, or, alternatively, disposed of by burial, if preferred. It may also be co-composted with other organic waste.

Sludge from septic tanks, aqua-privies, etc, can be disposed of only in accordance with the prescribed methods. See Water Research Commission (1997) *Permissable utilisation and disposal of sewage sludge.*

Unless the sludge has been allowed to decompose until no more pathogens are present, it may pose a threat to the environment, particularly where the emptying of pits is practised on a large scale. The design of facilities for the disposal of sludge needs careful consideration, as the area is subject to continuous wet conditions and heavy vehicle loads. The type of equipment employed in the disposal effort should be known to the designer, as discharge speed and sludge volume need to be taken into account. Cognisance should be taken of the immediate environment, as accidental discharge errors may cause serious pollution and health hazards.

Emptying facilities at treatment works need not be elaborate, and could consist of an apron on which to discharge the contents of the vehicle and a wash-down facility. The nature of the sludge can vary widely and this should be taken into account when designing the sewage works. Depending on the habits of the pit owners and the effectiveness of refuse removal in the area, there may also be a high proportion of rags, bottles and other garbage in the sludge. Generally a higher grit load will also come from developing communities. Pond systems can be very effective in treating sludge from on-site sanitation systems. If the ponds treat only sludge from pit latrines it may be necessary to add water to prevent the ponds from drying out before digestion has taken place. Sludge from on-site sanitation systems can also be treated by composting at a central works, using forced aeration.

Although it is usually still necessary to treat sludge from on-site sanitation systems, the cost of treatment is lower than for fully waterborne sanitation. This is because partial treatment has already taken place on the site through the biological decomposition of the waste in the pit or tank. In addition, the treatment works do not have to be designed to handle the large quantities of water which must be added to the waste for the sole purpose of conveying solids along a network of sewer pipes to the treatment and disposal works.

EVALUATION OF SITES

It is not possible to lay down rigid criteria for the suitability of a site for on-site sanitation, because soil and site conditions vary widely. Two basic criteria should, however, be considered – namely, whether the soil can effectively drain the liquids brought to the site and whether there is any danger of pollution of the groundwater or surface water.

Topographical evaluation

All features that can affect the functioning of the sanitation system should be noted and marked on the site plans during a visual survey of the site. The position of depressions, gullies, rock outcrops and other features should be noted and an assessment made of how they are likely to affect the functioning of the system. The type and gradient of slopes should also be noted as steep slopes can result in the surfacing of improperly treated effluent, especially during periods of high rainfall. Surface and subsurface drainage patterns, as well as obvious flood hazards, should be reported. The vegetation on the site will often reflect soil-drainage characteristics.

Soil profiles

Sampling holes should be excavated to a depth of at least one metre below the bottom of proposed pit toilets or soakaways. Soil properties (such as texture, structure and type) should be determined. The presence of bedrock, gravel, groundwater or a layer with poor permeability should be noted. Soil mottling indicates the presence of a high seasonal groundwater table, which can affect soil percolation.

Percolation capacity of the site

If the soil is unable to drain liquid waste effectively, swampy, unsanitary conditions can result. This can occur in areas with very shallow water tables or with poor permeability, or where a shallow, restrictive layer such as bedrock occurs. On-site sanitation can be used in low-permeability soils, but the system must be carefully selected in relation to the quantity of water supplied to the site. If the efficient functioning of a soil-percolation system is in doubt, it is advisable to create an inspection point where the level of the water in the subsoil drain can be monitored, so that adequate warning of failure is obtained to allow the local authority to plan for an alternative solution to liquid drainage problems before a crisis develops. This will be effective only if the recipient community fully understands the need for regular inspection of the drain, as well as the consequences of failure. On-site sanitation can be used in areas with poor percolation, but it may be necessary to retain all the liquids on the site in a sealed vault and provide a regular emptying service, or to drain the site with settled sewage systems.

The percolation test is designed to quantify the movement of liquids in the soil at a specific time of the year. Percolation rates usually change as the soil's moisture content changes, and it is best to conduct the test in the rainy season. The percolation test should be regarded only as an indication of the suitability of the soil for a specific sanitation system. The following procedures should be complied with:

Calculation of number of test holes

The number of test holes needed is determined by the size of the settlement and the variability of the soil conditions. Usually test holes should be spaced uniformly throughout the area, at the rate of five to ten holes per hectare, if soil conditions are fairly homogeneous.

Percolation test

The test procedure to be followed is described in SABS 0400.

POLLUTION CAUSED BY SANITATION

When there is a high density of people living in an area, both the surface water and groundwater can be expected to become polluted to some degree, irrespective of the type of sanitation system used in the area. Some basic precautions will minimise the risk of serious pollution.

Surface pollution

The surfacing of partially treated effluent can create a direct health risk, and can cause pollution of surface waters. This type of pollution should and can usually be avoided. It is most likely to occur in areas where the groundwater table is very high or in areas with steep slopes where a shallow, permeable layer of topsoil covers an impermeable subsoil. In areas where cut-and-fill techniques are used to provide platforms for house construction, sanitation units and soakaways should be carefully sited to minimise the possibility of surface pollution. Sanitation units and soakaways should also be sited in such a way that rainwater ingress cannot occur, as this could cause flooding, with resultant surface pollution.

Poor maintenance of reticulation systems, pumping stations and sewage-purification works can cause serious pollution with associated health risks, especially in remote areas close to streams and rivers.

Groundwater pollution

Groundwater can be contaminated by a sanitation system; therefore the risk should be assessed or the groundwater periodically monitored, particularly where this water is intended for human consumption. The guidelines made available in the publication *A protocol to manage the potential of groundwater contamination from on-site sanitation* by the Directorate of Geohydrology of the Department of Water Affairs & Forestry (1997), should be observed. The soil around the pit toilet or subsurface drain provides a natural purification zone, and tests carried out both in South Africa and other parts of Africa indicate that on-site sanitation does not pose a serious threat, provided the water is not intended for human consumption. Generally, the susceptibility of a water source to pollution decreases quite sharply with increasing distance and depth from the source of pollution, except in areas with fissured rock, limestone, very coarse soil or other highly permeable soils.

Soakaways attached to on-site sanitation systems should, wherever possible, be located downstream of drinking water supplies. The following could be used as a guide for the location of a soakaway:

- 7,5 m from the drinking-water source if the highest seasonal water table is more than 5 m below the bottom surface of the pit or soakaway;
- 15 m from the water source if the highest seasonal water table is 1-5 m below the bottom surface of the pit or soakaway;
- 30 m from the water source if the highest seasonal water table is less than 1 m below the bottom of the pit or soakaway; and
- there is no safe distance from a source of drinking water in areas that have fissured rock, limestone or very coarse soil.

These distances are given as a guide only. Permeability of the soil is not the only factor. Geology, topography, the presence of trees, groundwater flow direction, etc, also influence the position of the borehole (Xu and Braune 1995).

SULLAGE (GREYWATER) DISPOSAL

General

On-site excreta-disposal technologies require that separate provision be made for the disposal of sullage. Sullage, also referred to as greywater, is defined as all domestic wastewater other than toilet water. This refers to wastewater from baths, sinks, laundry and kitchen waste. Although this “greywater” is supposed not to contain harmful excreted pathogens, it often does: washing babies’ nappies, for example, automatically contaminates the water. Sullage, however, contains considerably fewer pathogenic micro-organisms and has a lower nitrate content than raw sewage. It also has a more soluble and biodegradable organic content.

Sullage is produced not only on private residential stands but also at communal washing places and taxi stands, and provision should therefore be made for its disposal.

Volumes

The per capita volume of sullage generated depends on the water consumption. The water consumption is to a large extent dependent on the level of water supply and the type of on-site sanitation the contributor enjoys. It is not difficult to find local figures of generation and one should try to obtain these, even if it requires actual measurement in the field. Typical figures are given in Table 10.2.

Health aspects

Mosquito breeding can take place where ponds are created by casual tipping of sullage, and conditions favourable for the development of parasitic worms could also be created in this way. Infection can also occur in constant muddy conditions. In order to reduce potential health hazards, it is of the utmost importance to choose the right option for sullage disposal. See also Figure 6.31, Chapter 6.

Disposal

The type of disposal system chosen by the designer will depend on various factors such as the availability of land, the volume of sullage generated per day, the risk of groundwater pollution, the availability of open drains, the possibilities of ponding and the permeability of the soil. Where water is available on the site, a disposal facility should definitely be considered.

Disposal systems

Sullage can be used to effect flushing of certain systems.

Casual tipping

Casual tipping in the yard can be tolerated, provided the soil has good permeability and is not continually moist. Where casual tipping takes place under other conditions it may result in ponding and/or muddy conditions, with adverse health effects as mentioned above. Good soil drainage and a low population density can accommodate this practice.

Garden watering

This practice can also be tolerated, provided plants and vegetables that are watered in this manner are not eaten raw, for disease transmission may occur.

Table 10.2: Sullage generation

AVAILABLE WATER SUPPLY AND SANITATION	SULLAGE GENERATION – LITRES PER CAPITA PER DAY
Standpipes, water vendors. Pit toilets.	20 - 30
On-site single-tap supply (yard connection). Pit toilets.	30 - 60

Soakaways

A soakaway is probably the safest and most convenient way of disposing of sullage, as long as soil conditions permit this. The design of the soakaway can be done according to the guidelines given in SABS 0400. Designers should be aware that groundwater pollution is still a possibility, though to a much lesser extent. Where simple maintenance tasks are able to be carried out, the use of grease traps should be considered.

Piped systems

The disposal of sullage in piped systems is hardly ever an economical solution, although it may be a viable option when dealing with communal washing points generating large amounts of sullage. Solids-free sewer systems are ideal for this purpose.

Sullage treatment

The fairly high BOD_5 value of sullage (typically 100 mg/ℓ) makes it unsuitable for discharging into rivers and streams. If treatment is required, single facultative ponds could be used.

TOILET PEDESTALS AND SQUATTING PLATES

Various types of toilet pedestals and squatting plates can be used with on-site sanitation systems. Members of some cultures are used to squatting for defecation, and cases of constipation have been recorded when they change to a sitting position. Some cultures also require water for anal cleansing.

The simplest form of appliance is a plain seat or pedestal, or a squatting plate. The hole should be approximately 250 mm in diameter for adults, and have a cover to restrict the access of flies and other insects to the pit. It is good practice to provide a second seat with a hole size of approximately 150 mm for young children, so that they need not fear falling into the pit. The plain seat found in a VIP toilet normally has similar dimensions.

Pedestals with water seals can be used in conjunction with most sanitation systems. Various methods can be employed to effect a water seal in a toilet system. They significantly increase user convenience by eliminating odour and screening the contents of the pit. Some water seal appliances require a piped water supply. These are not recommended for on-site sanitation systems, unless the extra water can be disposed of on the site (see the section on sullage disposal). Most water seal appliances can operate efficiently on a tank filled by a bucket of household wastewater.

A conventional toilet bowl is an example of a water seal pedestal, but it can require between 6 and 12 litres per flush. Special pans or bowls, so-called low-volume flush pans, have been developed that require only three litres per flush. These low-volume flush toilets do not have any negative effects on the self-cleaning capacity of waterborne sewerage systems. Various tipping-tray designs are also available, with flush requirements varying from 0,75 to 2 litres, depending on the design. These appliances have a shallow pan or tray that holds the water necessary for the seal. After use the tray is cleared by tipping it, allowing the waste matter to fall into the pit below. Thus the water is used solely for maintaining the seal, not for clearing the pan.

Pour-flush bowls can also be used to maintain a water seal. These pans are flushed by hand, using a bucket, and generally require about two litres per flush. The biggest disadvantage of this appliance is that the effectiveness of the flush depends on the human element – which varies greatly – and there is no control over the amount of water used per flush.

The pedestal of an aqua-privy does not have its own water seal. The water seal is effected by directing the pipe straight into the digester. No water is needed for flushing, but the level of liquid in the tank must be maintained.

DESIGN OF VIP TOILETS

Detailed design information can be obtained from the publication *Building VIPs* by Bester and Austin. A VIP Code of Practice is currently also under preparation by the SABS.

TOILET FACILITIES FOR PHYSICALLY DISABLED PERSONS

Some introductory remarks on water and sanitation facilities for disabled persons can be found under the section "Public or communal water-supply terminals" in Chapter 9.

Toilet aids help to preserve the dignity and independence of disabled persons. Outdoor toilets should be situated on smooth, even pathways, and ramps should be provided in lieu of steps. Ramps should be not less than 1100 mm wide, with a slope not exceeding 1:12. The appropriate layout and dimensions of a toilet room suitable for wheelchair users are shown in Figure 10.12. If there is enough space for a person using a wheelchair, then there should be enough space for ambulant people using crutches or technical aids. Taps or washbasins, if fitted, should also be in an accessible position and at an appropriate height for use from a wheelchair. Reference should be made to Part S of SABS 0400-1990 for further design information.

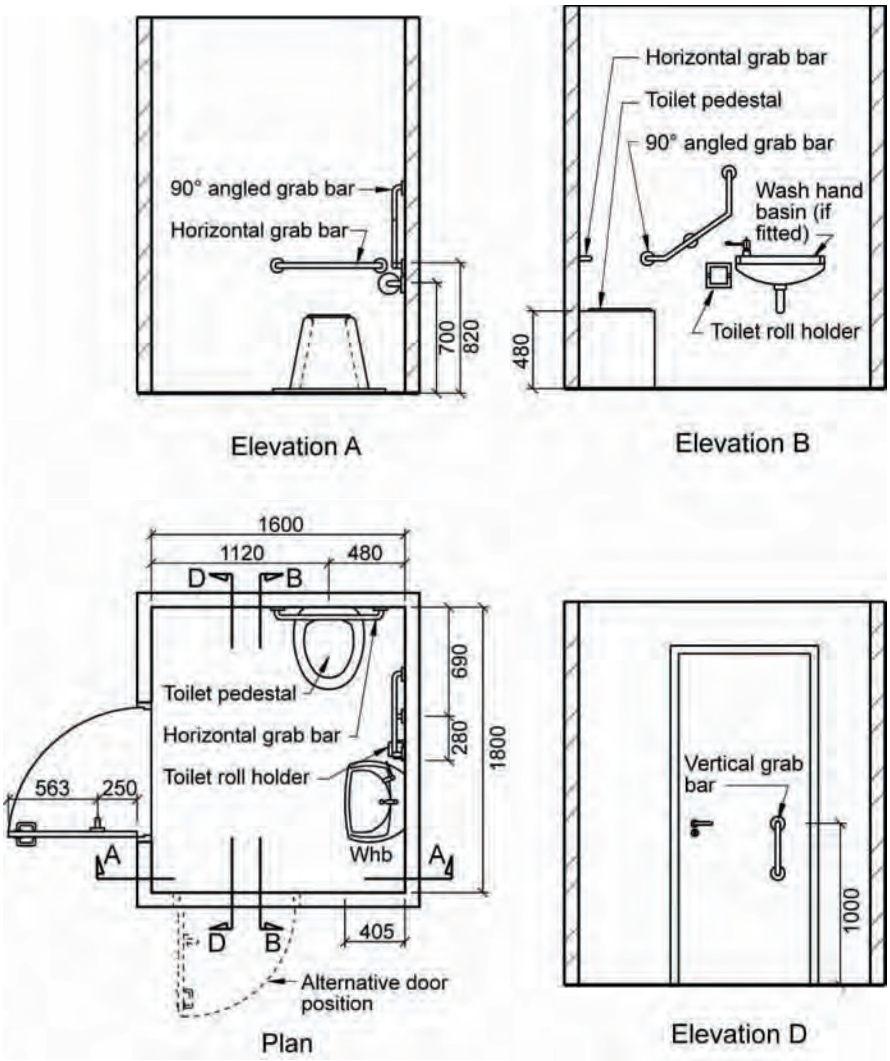

Figure 10.12: Minimum dimensions of a toilet room for physically disabled persons

TRAINING OF MAINTENANCE WORKERS

The training of local people as maintenance workers should be seen as an integral part of capacity-building within the community. Training should not take place merely because it is a fundamental principle or policy of the day.

Members of the community should be trained to install and operate a system, or to act as advisers to others who would like to install their own system. This can be achieved by establishing a "sanitation centre" where the community can purchase a variety of appliances and other necessary materials. The manager of this centre can be trained to become the local sanitation expert and can advise the population on maintenance requirements when necessary.

When assessing training needs, one must expect that some of the people trained will move to other areas where greater employment opportunities are available.

The degree of training and the amount of institutional support required will increase with the level of complexity of the sanitation system. On-site sanitation has the advantage that the greater part of the system belongs to the users. Sewer systems, on the other hand, have long lengths of underground piping and manholes, as well as pumping installations, which are the property of the local authority and must be regularly maintained by properly trained people in order for the system to work properly.

UPGRADING OF SANITATION FACILITIES

Possible upgrading routes should be considered when the initial choice of a system is made, particularly if an appreciable increase in the water supply is expected at some time in the future. However, it is unlikely that all the residents of a township will be able to afford upgraded services at the same time, which will be necessary if one is upgrading to full waterborne sanitation.

Most forms of pit toilet can be greatly improved by providing a water seal between the user and the excreta. This effectively stops odours and flies from exiting the pit via the toilet pedestal, and removes children's fears of falling into the pit. A water seal can be provided by a tipping-tray, a pour-flush or a low-flush toilet bowl. Designers should ensure that the quantity of water used with every cleaning operation does not increase the water content of the pit to a point where it can no longer be drained by the soil.

When the water supply is increased to the point at which the soil can no longer absorb it naturally (usually when an individual water connection is provided to each stand), it will be necessary to make special arrangements to remove the wastewater from each site in order to maintain healthy living conditions.

This is a major step in the upgrading process and will require additional financial input from the residents. The most economical solution may be a settled-sewage system, as this would be far easier to install in an existing settlement than a conventional sewer system.

If the developer is reasonably confident that the area's water supply will be upgraded within a few years, it may be advantageous to install septic tanks at the time of the original development. If this is done, then the settled-sewage system can be connected directly to the outlet from these systems and the soakaway can be bypassed and left inactive on the site.

Some possible upgrading routes are described in Appendix B.

OFF-SITE WASTEWATER TREATMENT

Off-site wastewater treatment is considered a specialised subject and, except for some general comments on pond systems and package purification units, falls outside the scope of this document. Where the introduction of a treatment works is considered, specialist consultants should be involved. The quality of effluent emanating from a wastewater treatment works is prescribed by legislation and has to meet licensing requirements. Water Services Providers should be licensed in terms of the National Water Act.

Pond systems

Although pond systems are regarded as treatment plants, the effluent does not normally meet acceptable effluent standards. Pond effluents have therefore to be irrigated. A pond system is regarded as a wastewater treatment works and its owner should also obtain a licence from the Department of Water Affairs & Forestry.

Pond systems are usually used in remote or developing areas, normally where land is available and relatively cheap. Skilled operators are not required and, depending on the circumstances, electricity need not be a requirement. Stabilisation (or oxidation) ponds are cheaper to build than conventional sewage purification works.

Although pond systems are regarded as being comparatively less sophisticated than other purification systems, they nevertheless require proper planning, application, design, construction and maintenance. Ponds do not need daily attendance, but should never be allowed to fall into disrepair.

The Water Institute of South Africa (WISA) has made available a design manual based on South African experience. This manual can be used as a guide to the design of pond systems.

Stabilisation or oxidation ponds are classified according to the nature of the biological activity taking place, as follows:

- facultative-aerobic ponds (where aerobic and facultative conditions exist) – facultative organisms use dissolved oxygen when it is available, but convert to anaerobic processes in its absence; and
- anaerobic-aerobic ponds (where the primary ponds are completely anaerobic and the secondary ponds are mainly aerobic).

The following important aspects should be considered regarding siting and land requirements:

- the cost of the land;
- the minimum distance between pond systems and the nearest habitation;
- the direction of the prevailing winds – ponds should, as far as possible, be downwind of town limits;
- possible groundwater pollution;
- geotechnical conditions that will influence costs;
- the land required for irrigation purposes, which is an integral part of the pond system; and
- the topography of the site, which can influence costs.

Irrigation of crops may take place only as prescribed in the publication *Permissable utilisation and disposal of sewage sludge* by the Water Research Commission 1997.

Package purification units

The use of package purification units is dependent on factors similar to those mentioned above, but the operational costs involved when opting for package purification plants should be carefully considered. It is not a preferred option.

APPENDIX A

INFORMATION REQUIRED FOR THE PLANNING AND DESIGN OF SANITATION PROJECTS

This appendix gives information regarding data that should be collected for the proper planning, design and implementation of a sanitation project. Note that the definition of sanitation in the White Paper on Basic Household Sanitation of 2001 is the following:

> "Sanitation refers to the principles and practices relating to the collection, removal or disposal of human excreta, household wastewater and refuse as they impact upon people and the environment. Good sanitation includes appropriate health and hygiene awareness and behaviour, and acceptable, affordable and sustainable sanitation services.
>
> The minimum acceptable basic level of sanitation is:
>
> (a) appropriate health and hygiene awareness and behaviour;
>
> (b) a system for disposing of human excreta, household wastewater and refuse, which is acceptable and affordable to the users, safe, hygienic and easily accessible and which does not have an unacceptable impact on the environment; and
>
> (c) a toilet facility for each household."

COMMUNITY MANAGEMENT APPROACH

Research done and projects launched in culturally different communities require an openness and adaptability to the issues that are important to the community. A growing awareness of the failures of conventional development approaches in meeting the needs of people with few resources has led to the exploration of alternative methodologies for investigating resource-management issues, and planning, implementing and evaluating development initiatives. There is a wide range of approaches with strong conceptual and methodological similarities. These include:

- Participatory rural appraisal (PRA);
- Participatory learning methods (PALM);
- Rapid rural appraisal (RRA);
- Rapid assessment procedures (RAP);
- Participatory action research (PAR);
- Rapid rural systems analysis (RRSA);
- The demand responsive approach (DRA);
- The SARAR approach;

and many others. The themes common to all of these approaches are:

- the full participation of people in the processes;
- the concept of learning about their needs and opportunities; and
- the action required to address them.

The experience gained in the past decade in particular has pointed to the need for three key elements to be successful both in water-supply and in sanitation projects, namely:

- involvement of the community in all aspects of the projects;
- the use of appropriate technology; and
- the need for institution-building and -training activities in conjunction with the project.

Community development, however, does not take place in a vacuum; it is always situated in a concrete social, economic and political context. Therefore, it is of the utmost importance that a multi-disciplinary team be involved in community development. Because development is development for the people, the people should remain central to the process. To ensure this, it is inevitable that social engineering should precede any development project, and run parallel with it until completion of the project.

Over the past few years, holistic development has become a vital aspect of sustainable development. It is recognised worldwide that projects that take human factors into consideration are more likely to be successful than those that do not. It is therefore of the utmost importance for development agencies to collaborate closely with communities at inception and through all stages of infrastructural development.

As distinct from community participation, community management means that beneficiaries of infrastructural services have the responsibility for, and authority and control over the development of such services.

Although the spin-offs of this approach are obvious, they are not easy to achieve within a short space of time. It should also be noted that the mere participation of communities in the project is not a solution, but a necessary forerunner of successful community projects. It is imperative that participation should be coupled with capacity-building efforts through training.

The application of scientific research methods (such as fact-finding and community surveys) is recommended in the initial stages of community development. These methods can be adapted in research forums that promote community participation, and which will create opportunities for interaction between the developer and the identification of needs by community members themselves. Examples include community surveys, social reconnaissance and action research. This emphasises a systems approach and inter-disciplinary teamwork, continued involvement of the community members in all decisions and activities, development as a learning process (including the need for training), and continued monitoring and evaluation activities.

Seven phases have been identified in the participatory strategy for integrated rural development.

Phase 1

This phase consists of an initial reconnaissance of the community among which the project is going to be implemented by the social scientist or community worker. Existing documentary sources about the community are studied and field visits, mini-surveys and interviews with key people, etc, are undertaken. The main aim is to identify initial goals and to commit the development committee, which must include representatives (male and female) from the community, to these goals.

Phase 2

The second phase is the identification of priorities by means of field studies and research. The planner/developer performs specific investigations in order to identify areas of priority or problems. Community members and other agents are trained in problem identification and analysis. Insight is also gained in the functioning of the system and its problems.

Phase 3

The third phase consists of the formulation of possible solutions for the identified problems. The social scientist or community worker gives direction, but community members are involved fully in exercises of discovering solutions. Continued involvement and participation of the community members should be ensured.

Phase 4

In the fourth phase feasibility studies are performed. It is important that objectives be compatible with each other.

Phase 5

This phase is the implementation of the project. The implementation implies various political and planning activities, such as official approval of the project, planning and design, formal project descriptions, communication with the authorities, liaison and linkage with other institutional agencies, financial support, physical input and specific services.

Phase 6

This phase consists of planning the completion, termination, or continuation of the project. Community members should be trained in the operation and maintenance of the system, and supported in their efforts for a period of time to ensure sustainability.

Phase 7

The seventh phase refers to project evaluation. Formal project evaluation, preferably by an external agent, is supported by internal evaluation procedures as an ongoing activity in the development process. Monitoring and auditing form part of the evaluation activities. The main function of evaluation is to identify weaknesses in a project, in order to avoid similar problems and facilitate sound planning in future projects.

HUMAN-RELATED DATA

The following data are necessary to ensure sustainability.

Socio-cultural data

Religious and tribal customs, as well as cultural factors affecting the choice of technology (for example, traditional materials and practices for cleansing and ablution), include:

- the general level of literacy and education, especially hygiene education;
- important watersource-related activities (such as laundry, bathing and animal watering); and
- community attitudes to the recycling and handling of decomposed human waste.

Community preferences

After considering costs, note preferences of the community regarding the following:

- the type of sanitation service;

- the position of the toilet (for example, should it be inside or outside the house? If outside, should the toilet door face the house? How far should the toilet be from the house?);
- the appearance of the toilet building and pedestal (colour, form, etc);
- the size of the toilet;
- the seats or squatting plates; and
- the permanency of the superstructure.

Economic and social conditions

These include:

- present living conditions, types of housing (including condition, layout and building materials used) and occupancy rates;
- population numbers according to income levels (present and projected), and the age and sex distribution of the community ethnic groups, and settlement patterns in the project area;
- land-use and land-tenure patterns;
- locally available skills (managerial and technical); and
- major occupations, approximate distribution, unemployment and under-employment.

Health and hygiene conditions

These include:

- location of toilet/defecation sites;
- toilet maintenance (structure and cleanliness);
- disposal of children's faeces;
- hand-washing and use of cleansing materials;
- sweeping of floors and yard;
- household refuse disposal;
- drainage of surrounding area;
- incidence and prevalence of water- and faeces-related diseases;
- treatment of diseases; and
- access to doctors/clinics/hospitals.

Institutional framework

These include:

- The identification and description (responsibility, effectiveness and weaknesses) of all institutions and organisations, both governmental and non-governmental, that are providing the following services in the project area:
  - water and sanitation;
  - education and training;
  - health and hygiene;
  - housing;
  - building material supplies; and
  - transport.
- Identification of all other major local organisations (social and political), the type and number of members they have and the influence they could have on the project.

Environmental and technical aspects

Important factors to consider are:

- the position of the site in relation to existing settlements;
- existing supplementary services (type, availability, reliability, accessibility, cost, etc) such as water supply, roads, energy sources and sanitation schemes;
- environmental problems such as sullage removal, stormwater drainage, refuse removal, transport routes;
- site conditions such as topography, geology (soil stability, rockiness, permeability, etc);
- groundwater data, such as availability, quality and use;
- prevailing climatic conditions such as rainfall, temperature and wind;
- type and quantity of local building materials that may be suitable;
- existing building centres supplying building materials and equipment (type of materials and equipment, availability, quality and cost); and
- the need for, and existence of, appropriate building regulations and by-laws.

INFORMATION REQUIRED FOR POST-PROJECT EVALUATION

To evaluate the success and sustainability of a sanitation project, it is necessary to correlate environmental conditions in the project area with the health profile of the community concerned after the scheme has been in operation for some time (especially projects where the upgrading of existing services is planned). To be able to do this, additional information should be collected – for example the following:

- A concise description of the community's living conditions in general, and their existing sanitation and water supply facilities in particular.
- How long have the people been living in the area and for what period have the existing sanitation facilities been in use?
- With regard to the community's perception of the present situation and sanitary practices, and their interest in or susceptibility to change:
  - Do they regard the present water supply as satisfactory, in quantity and quality?
  - What arrangements are there for refuse removal – do they regard them as satisfactory?
  - What facilities exist for personal hygiene – where do they bathe or wash themselves?
  - Are they aware of any advisory service on health and hygiene, and do they make use of it?
  - What amount of money do they spend on

    doctors, clinics, medicines and other health-related aspects?
  - Do they regard the present sanitation facilities as adequate?
- Identify all major health problems in the community:
  - List all diseases recorded in the area that can be related to water supply and sanitation.
  - Obtain figures on present morbidity and mortality rates.
  - Identify possible disease-contributing factors, such as possible contamination of drinking water.
  - What was the actual total cost of the project?

APPENDIX B

UPGRADING OF EXISTING SANITATION SYSTEMS

BASIC UPGRADING ALTERNATIVES

Alternative 1 – Aesthetic upgrading

The options in this category can be implemented by individual stand-owners, as the necessary financial resources become available. These are limited to the superstructure and the pedestal. An example would be where a functional superstructure is replaced with a more permanent or more aesthetic structure, such as one built from bricks and mortar. The property owner can decide on factors such as the size of the superstructure and whether the door should open inward or outward. The door and roof materials of the original superstructure could be re-used.

Alternative 2 – Introduction of a water seal

This will effectively remove odours emanating from the pit and will ensure that the user cannot see the excreta. This will also reduce the fear, particularly among children, of falling into the pit. A water seal can be introduced by installing a tipping tray, a pour-flush pan or a low-flush pan. The difference between the three types of water seal lies mainly in the quantity of water required, and thus their suitability would also depend on the ability of the soil to drain the additional quantity of water supplied to the site. This type of upgrading can be undertaken by the individual owner whenever funds are available. Depending on what type of water-seal appliance is used, it may or may not be necessary to provide an individual water connection to each stand.

Alternative 3 – Removal of liquids from the site

When individual water connections are provided to each stand, the situation will often arise where the soil can no longer adequately drain the additional water. This makes the removal of water necessary to maintain health standards. Liquids can be removed from the site by means of sewers (either a full waterborne sanitation system or a settled sewage system), or they may be retained on site in a conservancy tank and then removed periodically by a vacuum-tanker service. The installation of a sewer system will be a costly step in the upgrading process, and will require each resident to contribute to the construction costs, or some form of outside subsidy will need to be found. The ability of the local authority to manage and maintain these sanitation systems must be assessed when considering the introduction of the system. Generally, the sewers would need to be laid in the entire township at the same time. Upgrading should be undertaken only when the community can afford to pay for the higher level of service. Practically, this means that upgrading in Group 1 and 2 can be implemented at any time by individual property owners, independently of neighbours. On the other hand, upgrading in Group 3 will require both the greatest financial outlay from the property owners and implementation of the entire development at the same time. Note that, in the case of a settled sewage system, the sewers will need to be laid to neighbourhoods at the same time, but individual owners need not all connect to the system simultaneously, since it is not necessary to maintain cleansing velocities in sewers that convey only the liquid portion of the wastes. Thus, property owners could connect to the system when they have the financial means to construct the necessary solids-retention tank. Note that some on-site sanitation systems, for example septic tanks, can be used as solids-retention tanks and therefore can be connected to the settled-sewage system with only minor alterations.

UPGRADING ROUTES FOR THE VARIOUS SANITATION SYSTEMS

There are many possible upgrading routes that could be taken and the following should be seen as an indication of the various possibilities for the systems as defined in Table 10.1, which categorises sanitation systems.

Group 1

Upgrading chemical toilets

The use of chemical toilets would probably be a temporary solution in a developing community. Upgrading to a more permanent system would therefore take the form of total replacement with any one of the other sanitation systems. The chemical toilet would be removed from the site as a unit; thus there would not be any re-use of materials.

Group 2

Upgrading unventilated pit toilets

The first and most important step in upgrading would be to install a vent pipe to convert the toilet into a ventilated improved pit (VIP) toilet. This upgrading should be undertaken at the earliest possible opportunity. After the addition of a vent pipe, further upgrading would follow the same route as a VIP toilet.

Upgrading ventilated improved pit (VIP) toilets

The VIP toilet provides several opportunities for upgrading. A major improvement can be obtained by introducing a water seal between the user and the excreta, thus providing a level of convenience that is more acceptable to users.

It may thus be necessary to consider Alternative 3 upgrading (removal of liquids from the site) only if problems arise with the drainage of excessive quantities of water, which situation can be expected when individual water connections are provided to each site. Since the pit of a VIP toilet is not watertight, it will probably be necessary to construct a new tank on the site for solids retention if upgrading to a settled-sewage system is required. The pit of the VIP toilet will then become redundant. Thus, if at the outset the final stage of the upgrading route is known to be a conservancy-tank or settled-sewage system, it is preferable to begin with a sealed-tank system (such as a vault toilet, aqua-privy or on-site digester), to avoid constructing a new tank when the upgrading takes place.

The installation of a urine-diversion pedestal will make a significant difference to a VIP toilet. The contents of the existing pit should be covered with a layer of earth, and the structure may thereafter be operated as a normal urine-diversion toilet, where urine is diverted to a soakpit or collection container and faeces are covered with ash or dry soil.

Upgrading ventilated vault (VV) toilets

This system is a variation of the VIP toilet, with the important distinction that it has a waterproof pit or vault. The comments on upgrading in Alternatives 1 and 2, for VIP toilets, also apply to this system. Upgrading to Alternative 3 will be different from that of the VIP toilet because the VV toilet has a lined, waterproof vault that can be used. The option of upgrading to a conservancy tank is not mentioned because the ventilated vault toilet is a type of conservancy tank. Because the VV toilet already has a waterproof tank, this system is ideal for upgrading to a settled sewage system and it is therefore highly unlikely that this system would be upgraded to a fully waterborne sewer system.

Upgrading ventilated improved double pit (VIDP) toilets

This system is basically a variation of the VIP toilet, so the comments for the VIP also apply to the VIDP toilet.

Group 3

Upgrading full waterborne sanitation

No upgrading of this system is necessary, but the stand owner can implement aesthetic improvements to the pedestal and superstructure.

Upgrading conservancy tank systems

A conservancy tank provides an ideal opportunity for upgrading to a settled sewage system, since the tank can be used to retain solids on the site.

Upgrading the settled sewage system

No upgrading of this system is necessary, but the stand owner can implement aesthetic improvements to the superstructure.

Group 4

Upgrading septic tank systems

A septic tank also provides an ideal opportunity for upgrading to a reticulated system, since the outlet from the septic tank can be connected to a settled sewage system without any further alterations being necessary. Solids would be retained on the site and digested in the septic tank.

Upgrading aqua-privies

The aqua-privy has a rough water seal, but this can be greatly improved by removing the pedestal and chute and replacing them with a device such as a tipping-tray, pour-flush or low-flush pan. An aqua- privy also provides an ideal opportunity for upgrading to a settled sewage system, since the outlet from the aqua-privy can be connected into the reticulation system without any further alterations. Solids would then be retained on the site and digested in the aqua-privy tank.

APPENDIX C

DESIGN GUIDELINES FOR WATERBORNE SANITATION SYSTEMS

SCOPE

These guidelines are applicable to the design and construction of sewerage reticulation for undeveloped residential areas, where the future houses are to be provided with full waterborne sanitation. They do not apply to on-site drainage, and do not cover any form of on-site disposal such as septic tanks and soil-percolation systems. They also do not apply to settled sewage systems.

Certain basic guidelines applicable to non-gravity systems (i.e. pump stations and rising mains) are included, but detailed design criteria for these systems are not included, as they are regarded as bulk services.

Except in cases where illustrations are provided, the reader is referred to various figures in the relevant sections of SABS 1200.

DESIGN CRITERIA

Design flows

Flow-rate units

The unit of flow rate used in these guidelines is litres per second (ℓ/s).

Depth of flow and infiltration

Sewers should be designed to flow full at the peak design flow. An allowance of 15 per cent for stormwater infiltration and other contingencies should be incorporated in the design figures used for single-family dwelling units.

Average daily flow (A)

The average daily flow per single-family dwelling unit is given in Table C.1.

General residential
For erven zoned as “general residential”, including blocks of flats and hotels, an average daily flow of 600 litres per day for every 100 m^2 of erf size should be used.

Notes:
(i) The above figure is based on a dwelling unit with a floor area of 100 m^2 and a floor space ratio (FSR) of 0,6. If a FSR other than 0,6 is prescribed, the flow figure above should be adjusted accordingly.

(ii) Maximum density allowable under the scheme is the overriding factor.

Church sites
A church site should be treated as a “special residential” erf.

Schools and business sites
The discharge from day schools and business sites need not be taken into account, since these are relatively minor flows that do not peak at the same time as the main residential flow.

Peak design flows

In these guidelines the following factors apply to single-family dwelling units:

Peak Factor (PF) = 2,5
Percentage allowed for extraneous flow = 15 %

To calculate the unit design flow rate:

Average daily flow (ℓ/du/d) = A

$$\text{Average daily flow rate } (\ell/s) = \frac{A}{24 \times 60 \times 60} = \frac{A}{86\,400}$$

Table C.1: Average daily flows per single-family dwelling unit (du)

INCOME GROUP	LOWER	MIDDLE	HIGHER
Litres per dwelling unit per day	500	750	1 000
Based on average total persons per dwelling unit	7	6	5

Peak flow rate = Average daily flow rate x peak factor

$$\frac{A \times 2{,}5}{86\ 400} = B$$

Design flow rate = Peak flow rate + % of peak flow rate for extraneous flows

$$B \times 1{,}15 = \frac{A \times 2{,}5 \times 1{,}15}{86\ 400}$$
$$= 0{,}000\ 033\ A$$
$$= C$$

Thus, for a population up to 1 500,

$$C = \frac{A\ \ (\ell/s/du)}{30\ 000}$$

Thus, from Table C.1:

C = 0,0167 ℓ/s/du for lower income group
= 0,0250 ℓ/s/du for middle income group
= 0,0333 ℓ/s/du for higher income group

If unit design flows are, instead, obtained from actual flow-gauging of adjacent settlements of similar nature, these unit design flows should not exceed those given above.

Attenuation

To take advantage of the attenuation of peak flows in gravity sewer systems as the contributor area and population increases, design peak factors may be reduced in accordance with the graph in Figure C.1 for sizing any sewer receiving the flow from a population greater than 1 500. If actual local attenuation factors are available, however, these should be used instead.

Hydraulic design

Flow formulae

The following flow formulae are acceptable for the calculation of velocity and discharge in sewers:

Manning	(n = 0,012)
Crimp and Bruges	(n = 0,012)
Colebrook-White	(Ks = 0,600)
Kutter	(n = 0,012)

Any formula can be used as long as it produces values approximately the same as the equivalent Colebrook-White formula using Ks = 0,6.

Minimum size of sewers

The minimum diameter of pipe in sewer reticulation should be 100 mm.

Limiting gradients

Sewers may follow the general slope of the ground, provided that a minimum full-bore velocity of 0,7 m/s is maintained.

Table C.2 shows the minimum grades required to achieve this minimum full-bore velocity for various pipe sizes up to 300 mm in diameter.

If flatter grades and lower velocities than those in

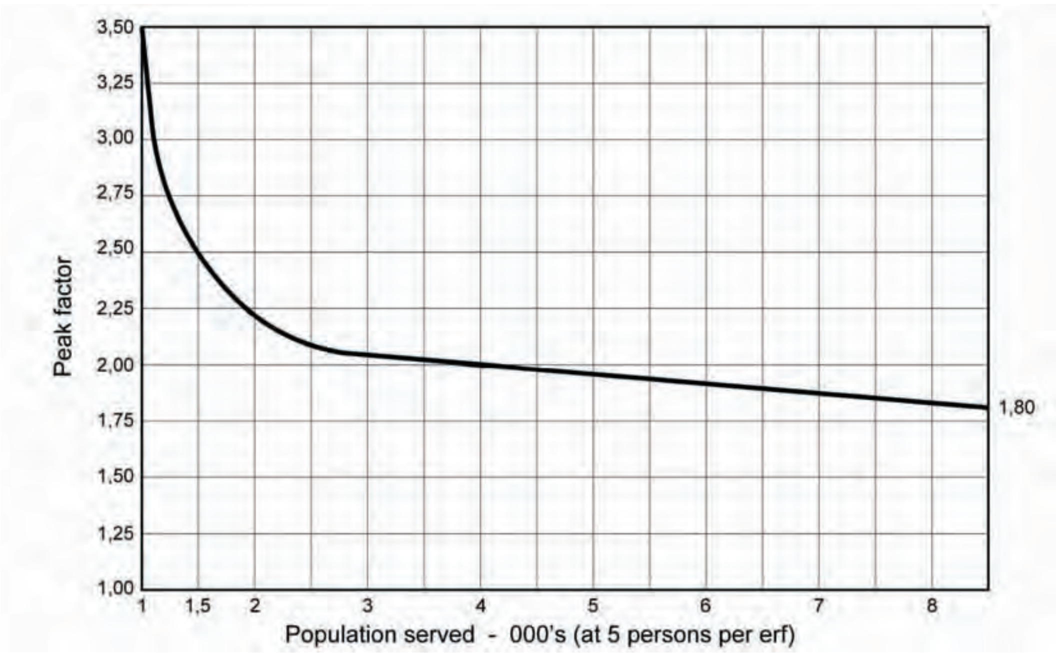

Figure C.1: Attenuation of peak flows

Table C.2: Minimum sewer gradients

SEWER DIAMETER (MM)	MINIMUM GRADIENTS
100	1 : 120
150	1 : 200
200	1 : 300
225	1 : 350
250	1 : 400
300	1 : 500

Table C.2 are contemplated, it is essential that a detailed cost-benefit study be carried out. This should take into account the cost of the regular systematic maintenance and silt/sand removal that will be required when flatter grades and lower velocities are used, instead of the additional first cost required to maintain the above minimum grades and full-bore velocity of 0,7 m/s.

Non-gravity systems

Rising mains
Velocities:
The minimum velocity of flow in a rising main should be 0,7 m/s.

The maximum velocity of flow in a rising main should be 2,5 m/s.

Minimum diameter:
The minimum diameter of a rising main should be 100 mm, except where a macerator system is used, in which case the diameter can be reduced to 75 mm.

Gradient:
Wherever practicable, rising mains should be graded so as to avoid the use of air and scour valves.

Stilling chambers:
Stilling chambers should be provided at the heads of all rising mains, and should be so designed that the liquid level always remains above the soffit level of the rising main where it enters the chamber. Stilling chambers should preferably be ventilated.

Sumps for pump stations
Emergency storage:
A minimum emergency storage capacity representing a capacity equivalent to four hours flow at the average flow rate should be provided, over and above the capacity available in the sump at normal top-water level (i.e. the level at which the duty pump cuts in). This provision applies only to pump stations serving not more than 250 dwelling units. For pump stations serving larger numbers of dwelling units, the sump capacity should be subject to special consideration in consultation with the local authority concerned. Emergency storage may be provided inside or outside the pump station.

Sizing:
In all pump stations, sumps should be sized and pump operating controls placed so as to restrict pump starts to a maximum of six per hour.

Flooding:
Care should be taken in the design of pump stations in order to avoid flooding of the dry well and/or electrical installations by stormwater or infiltration.

Screens:
Adequate protection, where necessary, in the form of screens or metal baskets, should be provided at the inlets to pump stations for the protection of the pumping equipment.

Pumps
Standby:
All pump stations should be provided with at least one standby pump of a capacity at least equal to the capacity of the largest duty pump. The standby pump should come into operation automatically if a duty pump or its driving motor fails due to mechanical failure.

Safety precautions
Safety precautions in accordance with the relevant legislation should be incorporated into the design of all pump stations and, in particular:

- all sumps and dry wells should be adequately ventilated;
- handrails should be provided to all landings and staircases and to the sides of open sumps and dry wells;
- skid-proof surfaces should be provided to all floors and steps; and
- the layout of the pumps, pipework and equipment should allow easy access to individual items of equipment without obstruction by pipework.

Physical design

Minimum depth and cover

Except under circumstances discussed in the following paragraph, the following are the recommended minimum values of cover to the outside of the pipe barrel for sewers other than connecting sewers:

•	in servitudes	600 mm
•	in sidewalks	1,4 m below final kerb level
•	in road carriageways	1,4 m below final constructed road level

Lesser depths of cover may be permitted, subject to integrated design of all services including trunk services allowed for in development plans, provided that, where the depth of cover in roads or sidewalks is less than 600 mm, or in servitudes less than 300 mm, the pipe should be protected from damage by:

- The placement of cast-in-situ or precast concrete slab(s) over the pipe, isolated from the pipe crown by a soil cushion of 100 mm minimum thickness. The protecting slab(s) should be wide enough and designed so as to prevent excessive superimposed loads being transferred directly to the pipes (see Figure C.2); or

225 mm diameter. Standard rigid pipes are laid on either Class D or Class C beds, as depicted in Drawing LB-1 of SABS 1200 LB, while flexible pipes (plastic or pitch fibre) are laid according to Drawing LB-2 of SABS 1200 LB.

Structural design of the pipe/bedding should be checked where trenches are:

- located under roads;

- deeper than 3 metres; and

- other than those classified as "narrow" (i.e. where overall trench width is greater than nominal pipe diameter d + 450 mm for pipes up to 300 mm diameter).

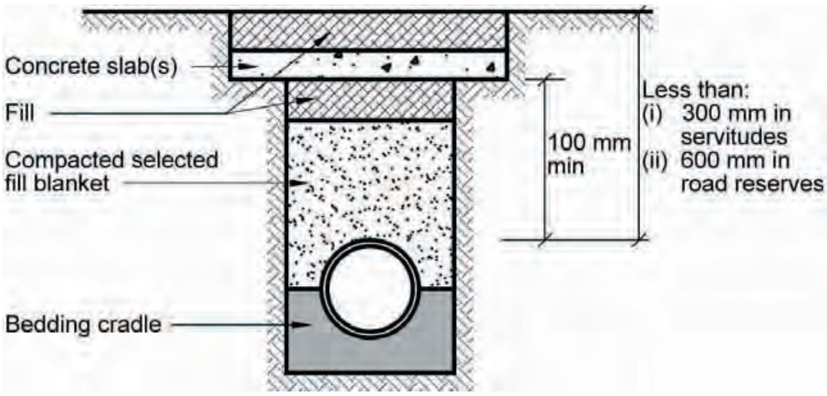

Figure C.2: Protection of pipes at reduced depths of cover (e.g. Class B bedding)

- The use of structurally stronger pipes able to withstand superimposed loads at the depth concerned; or

- The placement of additional earth filling over the existing ground level in isolated cases where this is possible.

Except in very special circumstances, the encasement of pipes in concrete is not recommended. Where encasement is unavoidable, it should be made discontinuous at pipe joints, so as to maintain joint flexibility (see Figure LD-6 of SABS 1200 LD).

Trenching, bedding and backfilling

The trenching, bedding and backfilling for all sewers should be in accordance with the requirements of SABS 1200 LB and the supporting Specifications.

Under normal ground conditions, structural design considerations for pipe strength and increased bedding factors do not come into play for sewers up to

Where grades steeper than 1 in 10 are required, 15 MPa concrete anchor blocks should be provided that are at least 300 mm wide and embedded into the sides and bottom of the trench for at least 150 mm, as shown on Drawing LD-1 of SABS 1200 LD.

Curved alignment

A straight alignment between manholes should normally be used, but curvilinear, horizontal or vertical alignment may be used where the economic circumstances warrant it, subject to the following limitations:

- the minimum radius of curvature is 30 m;

- curvilinear alignment may be used only when approved flexible joints or pipes are used;

- in the construction of a steep drop, bend fittings may be used at the top and bottom of the steep short length of pipe, thus providing a curved alignment between the flat and steep gradients.

Siting

Sewers should be sited so that they provide the most economical design, taking the topography into account (i.e. in road reserves, servitudes, parks, open spaces, etc). When the sewer is to be located in a trench by itself, the minimum clear width to be allocated to it in the road reserve should be 1,5 m.

Manholes

Location and spacing
Manholes should be placed at all junctions and, except in the case of curved alignment and at the top of shallow drops, at all changes of grade and/or direction.

The maximum distance between manholes on either straight or curved alignment should be:

- 150 m where the local authority concerned has power rodding machines and other equipment capable of cleaning the longer lengths between manholes;

- 100 m where the local authority concerned has only hand-operated rodding equipment.

Note:
The economics of acquiring power cleaning equipment in order to permit a greater manhole spacing should be demonstrated to local authorities.

Where manholes have to be constructed within any area that would be inundated by a flood of 50-years recurrence interval, they should, wherever practicable, be raised so that the covers are above this flood level.

Sizes
The minimum internal dimensions of manhole chambers and shafts should be as shown in Table C.3. The minimum height from the soffit of the main through pipe to the soffit of the manhole chamber roof slab, before any reduction in size is permitted, should be 2 m.

Benching
An area of benching should be provided in each manhole so that a man can stand easily, comfortably, and without danger to himself, on such benching while working in the manhole.

Table C.3: Minimum internal dimensions of manhole chambers and shafts

SHAPE	CHAMBER	SHAFT
Circular	1 000 mm	750 mm
Rectangular	910 mm	610 mm

Manhole benching should have a grade not steeper than 1 in 5 nor flatter than 1 in 25, and should be battered back equally from each side of the manhole channels such that the opening at the level of the pipe soffits has a width of 1,2 d, where d is the nominal pipe diameter.

Design
All manholes, including the connection between manhole and sewer, should be designed in accordance with the requirements of SABS 1200 LD and, where manholes are of cast-in-situ concrete, chambers, slabs and shafts should be structurally designed to have a strength equivalent to a brick or precast concrete manhole.

For manholes located in road reserves, spacer rings or a few courses of brickwork should be allowed for between the manhole roof slab and the cover frame, in order to facilitate minor adjustments in the level of the manhole cover. Adjustable manhole frames may also be used.

Steep drops
Steep drops should be avoided wherever possible, but where this is unavoidable (e.g. to connect two sewers at different levels), use should be made of a steep, short length of pipe connected to the higher sewer by one or more 1/16 bends and to a manhole on the lower sewer also by one or more 1/16 bends, as shown in Figure C.3.

Sewer connections

Size and siting
Each erf, excepting those listed below, should be provided with a 100 mm (minimum) diameter connecting sewer, terminating with a suitable watertight stopper on the boundary of the erf or the boundary of the sewer servitude, whichever is applicable. The connecting sewer should be located deep enough to drain the full area of the erf portion on which building construction is permitted.

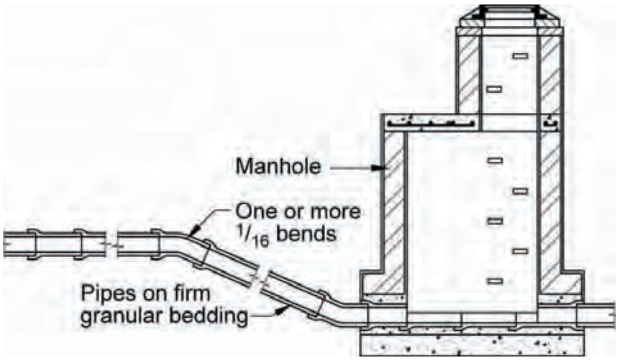

Figure C3: Steep drops in sewers

Exceptions

- In special residential areas, where an erf extends for a distance of more than 50 m from the boundary to which the connecting sewer is laid, provision need only be made to drain the area of the erf within 50 m of this boundary.

- School sites should be given special consideration with regard to the position, diameter and depth of the connection(s) provided.

- Where detailed development proposals are submitted for subdivided erven as group schemes, one connecting sewer may be provided to serve such group of erven.

Note:
Where erven have to be connected to a sewer on the opposite side of a street, consideration should be given to the economics of providing 100 mm diameter sewer branches across the road to serve the connecting sewers from two or more erven.

The sewer connection should be provided at the lowest suitable point on the erf. On street boundaries the connection should be located either at a distance of 1,15 m or at a distance of 5 m or more from a common boundary with an adjacent erf, unless a local authority has already an accepted standard location.

Depth and cover
Except under the circumstances described in the following paragraph, recommended minimum values of cover to the outside of the pipe barrel for connecting sewers are:

- in servitudes 600 mm
- in road reserves 1 000 mm

Where lesser depths of cover are permitted, this should be subject to the same conditions discussed previously in this appendix, and the same protection should be provided.

When designing the invert depth of the main sewer in order to ensure that all the erven can drain to it, the fall required from ground level at the head of the house drain to the invert of the main sewer at the point where the connecting sewer joins the main sewer should be taken as the sum of the following components:

- 450 mm to allow for a minimum cover, at the head of the house drain, of 300 mm, plus 150 mm for the diameter and thickness of the house drain;

- the fall required to accommodate the length of the house drain and the connecting sewer, assuming a minimum grade of 1 in 60 and taking into account the configuration of the erf and the probable route and location of the house drains; and

- the diameter of the main sewer (see Figure LD-7 of SABS 1200LD).

Note:
In the case of very flat terrain, and where the house drains may be laid as an integral part of the engineering services, flatter minimum grades than 1 in 60 for the house drains may be considered. This relaxation could also be applied to isolated erven difficult to connect, or the ground in such erven could be filled to provide minimum cover to the drains.

Junction with main sewer
A plain 45° junction should be used at the point where the connecting sewer joins the main sewer. Saddles should not be permitted during initial construction.

Type details
Details of the connecting sewer should be in accordance with one of the types shown in Figures LD-7 and LD-8 of SABS 1200LD.

Invert levels

The invert levels indicated at a manhole location should be the levels projected at the theoretical centre of the manhole by the invert grade lines of the pipes entering and leaving such manhole. In cases where branch lines with smaller diameters enter a manhole, the soffit levels of these branch lines should match those of the main branch line. However, in areas where pipes are laid to minimum grades, this practice may need to be relaxed.

The slope of the manhole channel should be as required to join the invert levels of the pipes entering and leaving the manhole, without allowing any additional fall through the manhole chamber.

MATERIALS

Pipes and joints

Pipes suitable for the conveyance of sewage, under the particular working and installation conditions to which they will be subjected, should be in accordance with Sections 3.1 and 3.2 of SABS 1200 LD.

All joints for rigid pipes should be of a flexible type, and rigid joints should only be used where the pipes themselves are flexible.

Manholes

All materials used for manholes should be in accordance with Section 3.5 of SABS 1200 LD.

Pumping installations

In general, all materials should be durable and suitable for use under the conditions of varying degrees of corrosion to which they will be exposed.

Pipework

The relevant requirements for materials given in SABS 1200 L and 1200 LK should apply if a rising main forms part of the sewerage system.

Concrete

Structural reinforced concrete and plain concrete below ground level and/or in contact with sewage should be designed and constructed in accordance with SABS 1200 G or 1200 GA, whichever is applicable.

Structural steelwork

All exposed steelwork should be adequately protected against corrosion with a suitable approved paint system, and should otherwise be designed and constructed in accordance with SABS 1200 H or 1200 HA, whichever is applicable.

Electrical installations

All electrical installations should comply with the Factories Act and with the relevant local authority electricity supply by-laws/regulations.

Other materials

Other materials used should comply with the requirements of SABS 1200 LD where relevant.

GLOSSARY

BOD_5: The oxygen used for bacterial oxidation of organic pollutants or ammonia, determined under standard conditions of incubation at 20° C over 5 days.

Sullage: Wastewater emanating from baths, kitchen sinks, laundries and showers (toilet water is excluded).

Thermophilic bacteria: A kind of bacteria functioning best at a certain temperature range.

Thixotropic: Refers to the property of becoming temporarily liquid when disturbed and returning to its original state when stationary.

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Chapter 11

Solid waste management

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

Aim of the guidelines

This chapter discusses waste management for developing urban areas. The aim of the guidelines is to assist such areas in implementing a waste-management plan that will enable them to deal with waste as economically and safely as possible. Waste produced by any urban community may, if left uncontrolled, not only be an aesthetic problem, but also pose serious health risks. This can be aggravated if hazardous material is present in the waste. It is therefore important that waste is collected from all sources as efficiently as possible, and disposed of in controlled disposal facilities. In the context of urban development, waste management at landfill sites is considered a bulk service and will not be discussed in any detail. As there are a number of existing disposal facilities in operation, it is necessary to understand the importance of proper disposal and the influence of landfill sites on the service provided and the community as a whole. The level of service is dependent on financial inputs and can therefore vary. There is, however, a basic level of waste management that needs to be provided to all communities. These guidelines should assist authorities to achieve this basic level and also provide some information to enable standards to be upgraded.

The waste cycle

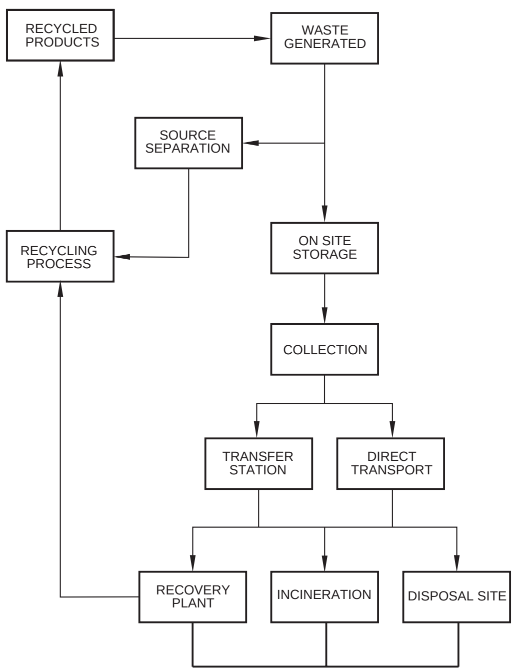


Figure 11.1 The waste cycle

As waste management comprises many variable and interrelated components, it is important to understand the waste cycle and that when one component of the system changes, it invariably affects other parts of the system.

Basic components

Generation

Should any changes at this level - such as source separation - be effected, consideration must be given to

- appropriate type and capacity of on-site storage required;
- changes to collection procedures;
- reduction in landfill volumes; and
- sustainable markets for recyclable products.

On-site storage

Any changes to on-site storage options mean that consideration must be given to

- waste volumes;
- waste types; and
- collection vehicles.

Collection

Any changes in collection vehicles will have an impact on

- the number of working crews required and therefore job opportunities;
- the capital and operational costs;
- the location of landfill and/or transfer stations; and
- the type of on-site storage.

Transfer stations/systems

Regardless of their degree of sophistication, transfer stations can

- assist in the reduction of haulage costs;
- reduce the congestion of traffic at the landfill; and
- provide opportunities for recycling.

Incineration

Large-scale incineration is capital-intensive, but has the advantage of

- reducing the volume of waste needing final disposal, with a resultant reduction in land use;
- combating the spread of disease; and
- providing a potential energy source.

Recycling

Before considering a recycling programme it is necessary to consider

- the involvement of the general public;

- recycling agencies;
- entrepreneurial development;
- education of the community;
- secure markets; and
- economic viability.

Disposal

With landfilling being the final step in the waste management cycle, consideration should be given the method used (i.e. baling) which would

- reduce cover material requirements;
- reduce both wind-blown litter and vermin;
- reduce leachate production; and
- influence the type of landfill equipment needed on the site.

It is therefore important to consider the implications prior to implementation, if sustainability of the service is to be achieved.

SOUTH AFRICAN SCENARIO

History

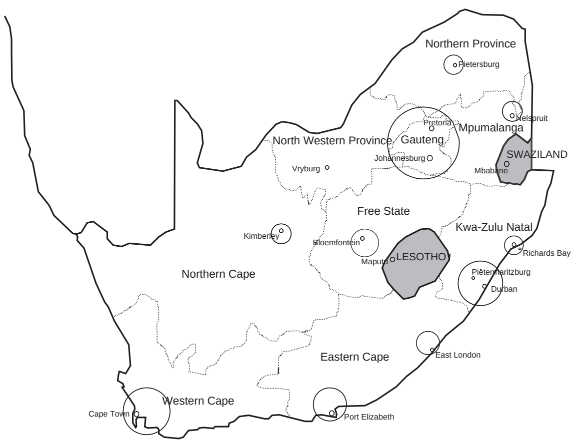


Figure 11.2: Major urban development areas

A formal waste collection service was first implemented in the Cape Colony in 1786, and by the 1820s a regular waste collection service on specific days of the week, using animal-drawn carts, was established.

It was only in the 1920s, with the advent of motor vehicles in South Africa, that the advantages of mechanical transport could be tested. The first trucks used for refuse collection were able to replace a number of carts with a significant cost saving and the advantage of easier supervision.

Social revolution

The rapidly changing socio-political situation has meant that traditional mechanised methods of collection have had to be rethought to adapt to the changing times. Rapid urbanisation, population growth and the ability of people to pay for the service are major influencing factors. Improvements in community health and demands for a better service, coupled with environmental concerns, are factors that significantly impact on waste management.

Mix of well-developed and poorly-developed areas

Population influx has created many informal and congested pedestrian-only settlements in open spaces and on the peripheries of towns and cities. This has provided the authorities responsible for waste management in South Africa with new challenges.

To integrate well-developed and poorly-developed areas, engineers and town planners need to provide innovative and cost-effective methods of waste collection, while maintaining a standard acceptable to both the community and the environment. This integration has created a new platform where the involvement of the communities in planning waste management systems is crucial, if sustainability and acceptance of the system are to be achieved.

Demand for land

Influx and rapid urbanisation, plus social and political pressures, have put land at a premium in the city and town areas. A city landfill once thought of as being an acceptable distance from suburban housing now sits cheek by jowl with generally low-income - but politically vocal and influential - communities. The search for acceptable disposal sites within an economically viable radius of collection operations, becomes more and more problematic. Public participation and consultation is therefore of the utmost importance.

Social upliftment and empowerment in underprivileged areas

With about half the potentially economically active population unemployed, there is increased pressure on authorities to couple the delivery of social services with increasing elements of job creation. Labour-intensive - rather than mechanised - methods are positively encouraged by central government and this form of collection is rapidly becoming the norm rather than the exception, as it provides entrepreneurial opportunities and job creation in an activity where there are no serious technical or financial barriers to entry. The government's privatisation policy further encourages this pattern, but the demand for higher wages and career opportunities must mean that "old technology" must incorporate modern methods of cost control, efficiency and planning in order to provide a cost-effective service.

Public perceptions

The concept of waste management is generally understood by most citizens of South Africa. Discussions with residents in both formal and informal settlements from Slovoville west of Johannesburg to Mandela Village east of Pretoria, and as far afield as Emondlo in northern KwaZulu-Natal and Phuthaditjhaba in the Free State, highlighted the need for a regular collection service and adequate on-site storage facilities. There is also a huge plea for help from most communities to educate and assist them in understanding the fundamental principles of waste management.

Legislation

It is vital that the local authority, including its representatives and community leaders as well as the service provider or contractor, be familiar with all legislation regarding waste management. This is necessary to ensure that, regardless of the level of service implemented, an affordable and environmentally acceptable standard is achieved.

Table 11.1: Current South African legislation

ACT	MAIN REQUIREMENTS	CONTROLLING AUTHORITY
Environment Conservation Act 73 of 1989		
Sections 19, 20, 21, 22, 24, 25	These sections cover definitions, prohibitions, regulations, procedures for permit applications, regulatory powers, offences and penalties, forfeiture and delegatory powers	Department of Environment Affairs
Water Act 54 of 1956 (certain sections still in effect at time of writing)		
Sections 21(1), 22, 23, 26	These sections cover the prevention of pollution by effluent, stormwater control, location of waste sites, offences and penalties, policies and strategies	Department of Water Affairs & Forestry
Health Act 63 of 1977		
Sections 1, 20(1), 38, 39, 40, 57	These sections cover definitions, prevention of pollution of water for human consumption, regulations regarding communicable disease and relating to rubbish, nightsoil, nuisances, offences and penalties	Department of National Health & Population Development
Atmospheric Pollution Prevention Act of 1965		
Sections 1, 17, 18, 19, 20, 27, 28, 33	These sections cover definitions, prevention of burning, smoke control, smoke and dust control areas and regulations	Department of National Health & Population Development, in conjunction with Local Authorities
Occupational Health and Safety Act 85 of 1993		
Sections 1, 3, 4, 10, 11, 12, 13, 14, 15, 16	With particular reference to hazardous chemical substances these sections cover definitions, general duty of care, control of exposure, protection and maintenance of equipment, prohibitions, transport and storage, disposal, offences and penalties.	Department of Labour

WASTE CATEGORIES

Waste by definition can be described as any matter - whether gaseous, liquid or solid - originating from any residential, commercial or industrial area, which is superfluous to requirements and has no further intrinsic or commercial value.

Domestic and household waste

Domestic and household waste comes mainly from residential areas and may include foodstuffs, garden waste, old clothing, packaging materials such as glass, paper and cardboard, plastics, and, in certain cases, ash.

Business and commercial waste

Business and commercial waste from offices, stores, and schools consists mainly of packaging materials such as glass, paper and plastics, cans, etc, with a limited quantity of foodstuffs emanating from hotels and restaurants. Where smaller industries are dispersed among normal commercial operations, regular monitoring is necessary to identify the need for special collection and disposal procedures.

Sanitary waste

Although not considered part of the general waste stream, if no proper sanitation system exists arrangements for the controlled removal and disposal of sanitary waste must be provided for (see Chapter 10 on Sanitation).

Non-hazardous industrial waste

Non-hazardous industrial waste (excluding mining waste) generally consists of a combination of commercial waste and discarded metal, timber, plastic and textile offcuts. With the majority of industries placed within municipal boundaries, careful identification of these wastes to ensure that proper disposal procedures are followed is important, particularly where chemical processing is apparent.

Construction waste

Construction waste generally consists of inert materials such as rubble and bulky construction debris. If mixed with household waste it attracts rodents, which can constitute a health risk, but it is generally considered more of an aesthetic problem. Removal of this material can in some instances require specialised equipment.

Hospital and medical waste

Waste from hospitals is generally separated at source. The general component is collected with normal domestic refuse. The medical component, consisting of body tissue, discarded syringes, swabs and contaminated material, is normally incinerated on site or collected by specialist private concerns. With smaller clinics and doctors' rooms, where the risk of accidents is no less, special arrangements for the collection and disposal of contaminated waste is essential. Mini-incinerators are also commercially available.

Hazardous and toxic waste

Due to the complexity of identifying the proper handling and disposal procedures, hazardous and toxic waste is generally the domain of specialist operators. It is, however, important that industries producing hazardous and toxic substances are identified and monitored to ensure that proper procedures are followed.

ON-SITE STORAGE

Inadequate on-site storage and collection systems account for the bulk of illegally dumped refuse in a large percentage of developing communities. The method of on-site storage thus has a significant effect on the collection system to be implemented (see Figure 11.3). It is important to plan and decide on the appropriate means of on-site storage in conjunction with transport options before implementing any system.

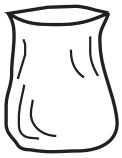
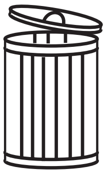
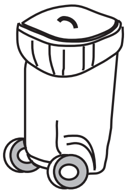
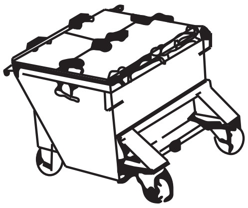
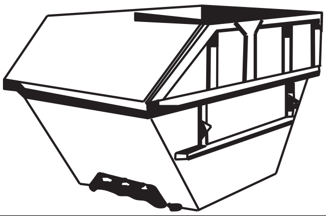
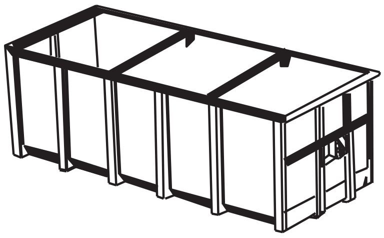
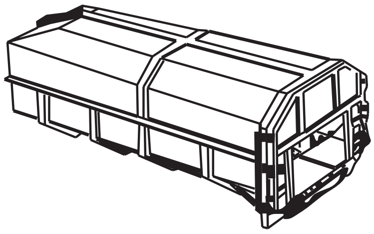
DESCRIPTION	COMMON USAGE		COLLECTION METHODS
85 litre plastic bin liners	Domestic/household Small business & industry General public amenities		By hand on-site or on sidewalk Liners deposited directly into collection vehicle
85 litre rubber/galvanised steel bins	Domestic/household Small business & industry General public amenities		By hand on-site or on sidewalk Bins emptied directly into collection vehicle
120/240 litre mobile refuse bins	Domestic/household Small business & industry General public amenities		Rear-end loading compactors with special lifting equipment
1,0 and 1,2 m^3 mobile refuse containers	Small business and industry		Rear-end loading compactors with special lifting equipment
4,5, 5,5, 6, 9 & 11,0 m^3 bulk containers	Large business & industry, garden refuse, building rubble, general public amenities & bulk wastes and communal collection systems		Load luggers Rear-end loading compactors with special lifting equipment
15 to 30 m^3 open bulk containers	Large business & industry, garden refuse, building rubble & bulk wastes and communal collection systems		Roll-on roll-off vehicles
11, 15 & 35 m^3 closed ontainers	Large shopping centres, transfer stations & selected industries		Roll-on roll-off vehicles

Figure 11.3: On-site storage options

WASTE COLLECTION

Local authorities are responsible for ensuring that a service is provided to the communities they serve. Collection can be done by the local authority, a conventional contractor, or an emerging entrepreneur. Several factors therefore need to be considered when selecting the appropriate waste management approach for a particular community, all of which will influence the waste handling and disposal options. These factors include:

1. Affordability
    - capital and operational costs;
    - level of income within the community; and
    - grants or subsidies available.
2. Accessibility
    - road infrastructure and conditions.
3. Level of education
    - literacy and awareness of the community to understand the principles of waste management.
4. On-site storage facilities
    - availability and suitability; and
    - composition and volume of the waste.
5. Potential benefits
    - clean and healthy environment; and
    - job creation and upliftment.
6. Available facilities and infrastructure
    - appropriate vehicles; and
    - available expertise.
7. Distance to disposal site
    - transfer facility requirements.
8. Pollution potential
    - blocked sewers and stormwater canals; and
    - illegal dumping and littering.

Collection systems

Many innovative or alternative collection systems have been attempted in South Africa, where collected waste is exchanged for food, or financial reward is made for bags collected. These have, in the main, been financed through grants or sponsored by various agencies and can therefore not be regarded as sustainable services, but rather as a process of education which should be supplemented by an effective collection system.

Communal collection

Communal collection is generally considered an option in poorly developed areas where the householders or entrepreneurial contractors are required to place the waste in strategically positioned containers for collection and disposal by large motorised refuse vehicles.

Door-to-door collection

Door-to-door collection can be carried out in two ways:

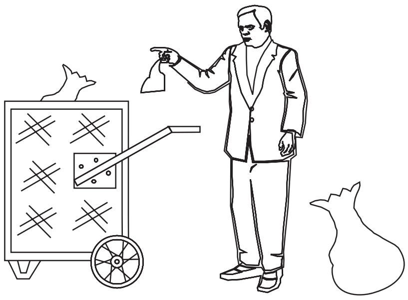
(a) the collection crew removes the waste container from the premises and returns it after it is emptied into the collection vehicle; and

(b) the householder places his or her refuse containers on the sidewalk, ready for collection by the collection crew, and retrieves them after collection.

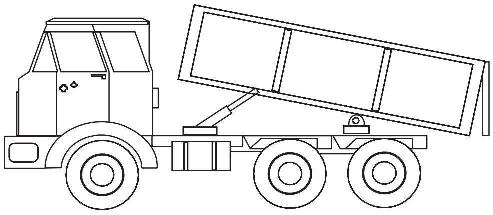
It is important to understand that flexibility of refuse collection is an important factor in today's urban development. Although it is understandable and logical that an efficient mechanical system of collection can evolve in a conventional suburban environment, this method may be totally inappropriate for highly dense, congested settlements that have mushroomed on the fringes of local authority areas. Now, more than ever, administrators and engineers are faced with a dilemma: capital-intensive solutions that were well justified a few short years ago, are becoming obsolete and inappropriate less than halfway through their amortisation period. The norm, for the next decade at least, will be to have a mix of old and new technology and the ability to integrate both systems to maximise economic advantage.

Collection vehicles

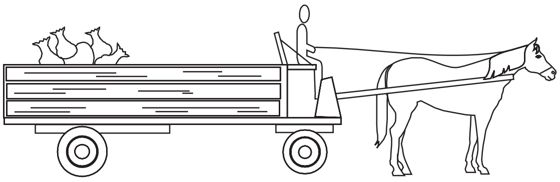
There are several options available for transporting collected waste for disposal, ranging from the basic hand cart to the technically sophisticated and motorised front- and rear-loading compaction vehicles. All options have a place in providing an effective collection service in the varied and mixed development areas currently faced by engineers and administrators. Careful consideration of the local road conditions, accessibility and topography - in conjunction with town planners - of the area to be serviced is needed before selecting any one of the options (see Chapters 5 and 7).



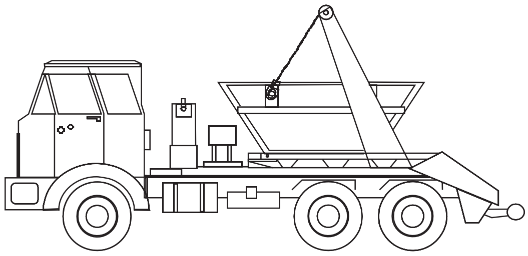
Push cart



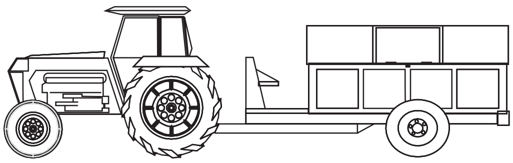
Tip truck



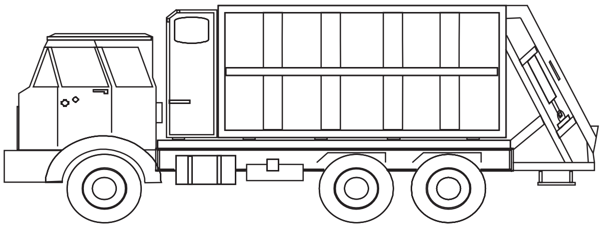
Animal-drawn cart



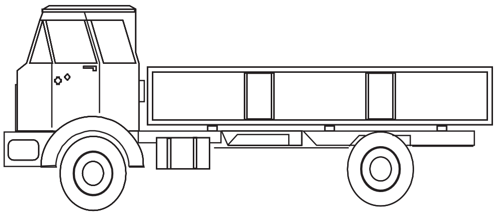
Load lugger (skip loader)



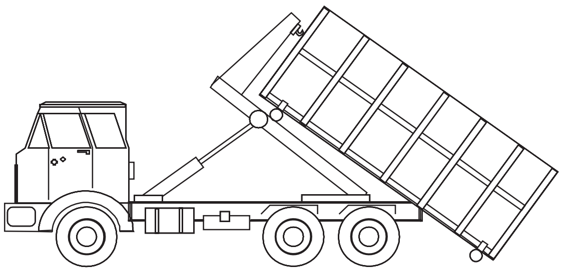
Tractor-trailer



Rear-end loader



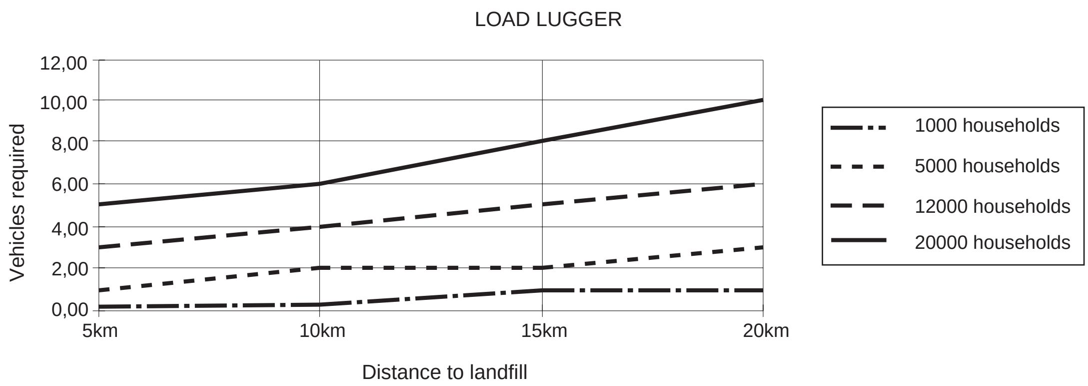
Flat-bed truck

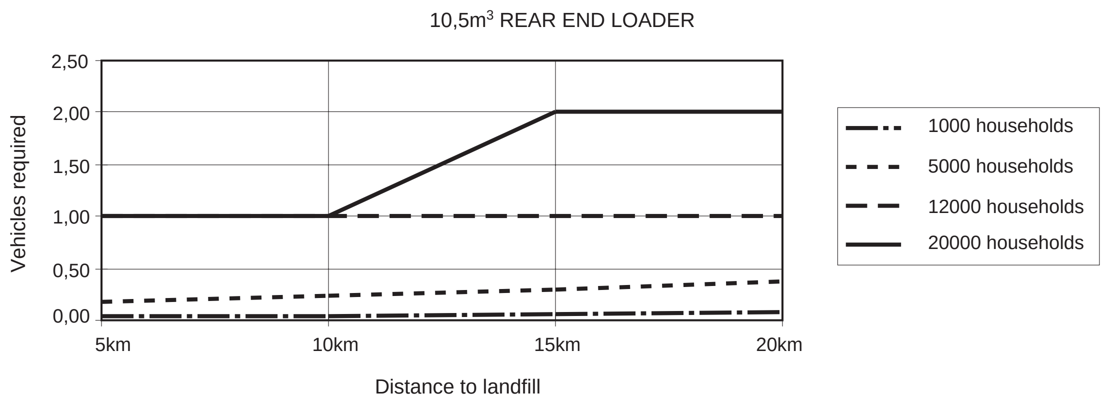


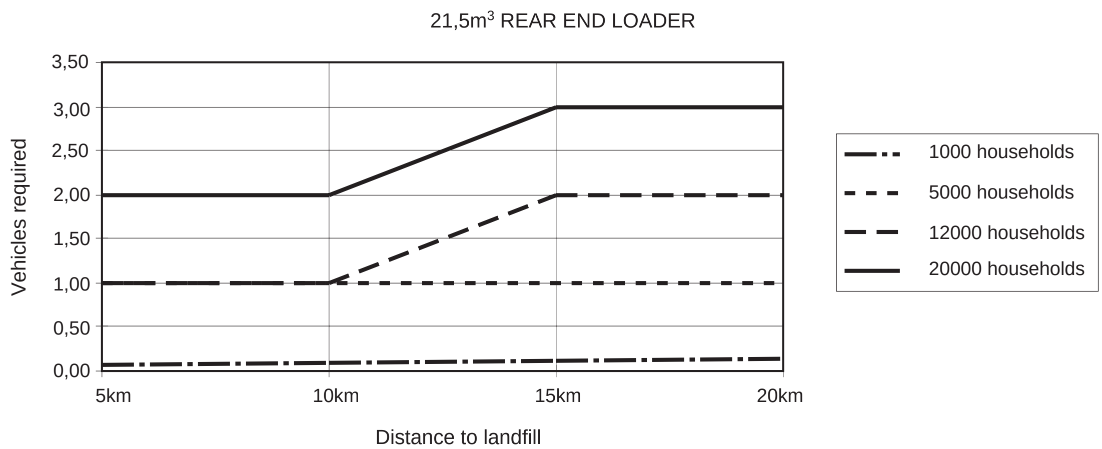
Roll-on roll-off

Figure 11.4: Refuse collection and transport options

The graphs below give an indication of vehicle requirements based on the relationship between distance to landfill / transfer station and number of households







Assumed

1. 0,12m^3 of waste per household per week
2. Average speed 40km/h

Figure 11.5: Graphs of vehicle requirements

Should more than one option be considered, it is equally important to identify how they will complement each other to eliminate the need for radical changes as the development progresses. The type of vehicle selected will also depend on the waste composition, as high-density waste (high ash content) will not require compaction. The relationship between payload and distance to the landfill or transfer station must also be considered. Table 11.2 gives some indication of the capital cost of the various options.

The options for transport are the following:

Hand cart

Hand carts, although not commonly used in South Africa, can be designed for specific applications. These may include small informal communities with no planned or designed road infrastructure, and even planned developments during the early stages where occupancy does not warrant sophisticated equipment, particularly in what can be considered the lower income groups. Although limited in carrying capacity, hand carts can be effectively employed where job creation and limited capital expenditure are the main considerations.

The use of hand carts has the advantage of not only providing more employment opportunities within the community due to the relatively small areas of responsibility of the operator, but also of combining street cleaning with normal refuse collection. The main disadvantage is that they would need to be supplemented with a communal bin system and the appropriate vehicle for final disposal.

Animal-drawn cart

Animal-drawn carts are also not commonly used in South Africa, yet have similar applications to the hand cart. The only significant difference is that the area of responsibility can be increased due to a relatively larger carrying capacity. A disadvantage is that animal-drawn carts can only be used if a community-based waste collection system is implemented and the "contractor" has his own choice of transport. Should the disposal facility be within a practically attainable distance, the need for support facilities could also be eliminated, provided access via freeways is not required.

Tractor and trailer

The tractor and trailer, although not the first, was probably the most common means of mechanical transport for waste collection prior to the introduction of compaction units, and can still be effectively utilised in most developed and undeveloped areas. The variations and combinations available are numerous and must be carefully assessed prior to implementation. The tractor-trailer combination can be operated where road conditions are not suitable for trucks, but is limited to a maximum 10 kilometre distance to the disposal facility, provided access via freeways is not required.

Flat-bed truck

The flat- or open-bed truck is not commonly used in South Africa. It has the disadvantage of a high loading height, particularly for use in developing areas where the high ash content of the collected waste results in increased mass of the container.

Table 11.2: Capital cost of transport and collection options (1996)

DESCRIPTION	UNIT	TRACTOR	TOTAL
19 m^3 half pack fel	348 500	400 000	748 500
19 m^3 F5000 rel	207 000	400 000	607 000
20,6 m^3 HC250 rel	175 600	400 000	575 600
12 ton roll-on roll-off	111 000	400 000	511 000
8 ton roll-on roll-off	85 000	400 000	485 000
12 ton skip	66 700	400 000	466 700
8 ton skip	64 700	340 000	404 700
10,5 m^3 M160 rel	110 000	250 000	360 000
20 m^3 gravity pack	68 200	250 000	318 200
13 m^3 gravity pack	33 400	250 000	283 400
aim refutip 20 m^3	145 000	100 000	245 000
dome trailer 12 - 15 m^3	60 000	100 000	160 000
power system	40 000	100 000	140 000
motorised tricycle			40 000
hand cart max 1,2 m^3			750

Waste must also be covered during transport to prevent further environmental nuisance.

Tip-pack

The tip-pack is the forerunner of the rear-end loader. With the tip-pack, the weight of the refuse is used for compaction and normally has a capacity not exceeding 10 m^3. Trailers with similar applications have since been developed for use with tractors.

Rear-end loaders

Rear-end loaders are available in sizes varying from 10 m^3 to 21 m^3, and have a relatively advanced compaction system allowing a compaction ratio of up to 4:1. This high compaction ratio and carrying capacity, with the versatility of being capable of handling containers up to 6 m^3 , has made the rear-end loader the most popular and commonly used collection vehicle in developed areas. This is particularly so where volumes are high and distances to disposal facilities are in excess of 10 km. The main disadvantages of using the rear-end loader are relatively high maintenance costs, and that they can only be effectively used where good road infrastructures are in place. Legal payloads need to be monitored in the event of high-density wastes, to prevent a negative impact on the road infrastructure.

Load luggers

The load lugger is a special application vehicle and limited to container applications. The most common application is the handling of bulk containers from industries and large businesses, communal collection systems and the removal of builders rubble from construction sites.

Roll-on roll-off

Roll-on roll-off vehicles are specially designed vehicles with very specific applications. They are mainly used for the transportation of large-capacity open or closed compacted containers ranging from 18 m^3 to 30 m^3.

Rail

Rail has only recently become an option in waste management, particularly where the pressures of urban development have dictated that landfills need to be positioned further from the source of waste generation.

STREET CLEANING

Wind-blown litter and illegal dumping of uncollected waste are probably the most visible aspects of poor waste management within any community and one that unfortunately receives the least attention, particularly in newly developed communities. If not controlled, these become major contributors to blocked stormwater drains and sewers, often the main cause of complaints within communities. Street cleaning is unfortunately an unrecoverable cost but a necessary component of the waste collection service that needs to be provided, and should be budgeted and planned for in any collection system (see also the section on street cleaning in Chapter 6: Stormwater Management).

TRANSFER STATIONS/SYSTEMS

A transfer station is a facility for transferring waste from the collection vehicle to a more appropriate vehicle where longer haul distances are necessary for final disposal. The purpose is to reduce not only the transport unit cost of collection vehicles, and obtain more cost-effective payloads, but also to allow quicker turnaround times and therefore increased productivity.

The need for a transfer station and the degree of sophistication required will be determined by the volume of waste generated, the collection system implemented, and the distance to the disposal site.

A transfer station can be designed to operate at various levels of sophistication:

- depositing the waste into containers of various capacities for specially designed vehicles, commonly used in communal waste-collection systems and garden-refuse sites;
- depositing the waste onto a suitably designed platform for either mechanical or manual loading into the long-haul vehicle;
- depositing directly into the long-haul vehicle; and
- depositing the waste into large compacting units for transporting by specially designed vehicles.

Transfer stations can be considered the final disposal point by the community, particularly where communal collection services are in operation. Communal disposal facilities, where open bulk containers are utilised, therefore need to be managed and controlled with the same care and responsibility as that required for a landfill site.

Location

The location of communal disposal points must be selected with sensitivity and careful planning to ensure accessibility and acceptance by the community, and not to interfere with pedestrian movement, or create an eyesore, or a public nuisance of dust and odour. Refer also to Chapter 5.

The larger or more sophisticated transfer station systems should be located with equally careful planning. Easy access to trunk routes and minimal conflict with future development are some of the more important considerations.

Other aspects often not considered in locating transfer stations and landfill sites, particularly in large operations, are traffic densities and road design. It is important that the requirements of both the local and provincial authorities are complied with, particularly in respect of paving design as well as visibility and traffic flow at intersections and access roads. Refer also to Chapters 7 and 8.

RECYCLING

Recycling means the remanufacturing of recovered materials, as opposed to re-use where the recovered product is simply re-used for similar purposes, e.g. beverage bottles.

The increasing pace of life in South Africa's developed commercial and industrial areas, particularly in the larger cities, reflects an increasing demand on the individual's time - both in work and leisure activities. These demands have changed consumption needs, which in turn have led to an increase in discarded goods and packaging.

Aim

The ultimate aim of recycling is the protection of the environment and public health by reducing the ever-increasing volumes of waste being generated by developing societies, as well as reducing the amount of natural resources necessary for the manufacture of any product.

History

Despite several million rands having been spent on sophisticated recycling plants, the history of recycling on a large scale in South Africa has not been particularly successful. There has been reasonable success in certain regions, with organisations such as Collect-a-Can, Nampac, Sappi, Mondi and Consol Glass concentrating mainly on beverage cans, paper, plastics and glass. Voluntary recycling and small buy-back centres have met with some success, which is generally dependent on the user-friendliness of the scheme.

Markets

The success of recycling is largely dependent on the market availability of both the raw product and the remanufactured product, and unfortunately they have a direct effect on each other. As waste cannot be considered recycled until it is reprocessed, these markets have achieved little more than providing subsistence levels for the individual entrepreneurs.

Education

Education in the principles and effect of waste minimisation and recycling is a critical part of the waste management process. The process of education will no doubt be slower in developed communities, where new attitudes need to be cultivated. The opportunities for success are far greater in newly developed communities if proper planning and educational programmes are an integral part of the collection system.

DISPOSAL

All communities, regardless of size and location, will need to make provision for the final disposal of collected refuse. To ensure environmental acceptability the Department of Water Affairs & Forestry (1994) has produced the document Minimum Requirements for Waste Disposal by Landfill. It is important to understand that, should no registered landfill site be available, a permit for disposal based on the "minimum requirements" will be enforced as required under Section 21 of the Environment and Conservation Act (Act 73 of 1989).

Table 11.3: Landfill classification

LANDFILL SIZE	MAX. RATE OF DEPOSITION (tons per day)
Communal	Less than 1
Small	Greater than 1; less than 25
Medium	Greater than 25; less than 500
Large	Greater than 500

Landfill site classification

The volume and content of the waste to be disposed of will dictate the size and classification of the landfill, and necessary requirements for licensing purposes. There are generally only two distinct types of landfill:

- general waste landfills (G) for waste normally produced within residential and business communities, with the exception of hazardous or toxic wastes; and

- hazardous waste landfills (H) for waste which has the potential to have adverse effects on both public health and the environment, even in small quantities, due to it's inherent chemical and physical qualities.

The process of developing any landfill site requires the local authority or private concern to conduct an investigation as to whether it complies with the minimum requirements under Section 4 (Site selection) of the above document provided by the Department of Water Affairs & Forestry. Should the results of these investigations meet with the Department's approval, it will be necessary for the developer to obtain a permit to develop and operate the landfill facility, based on further requirements outlined in Section 5 (Permitting) of the same document.

INCINERATION

With large open tracts of land still available and the high capital cost of this option, waste incineration on a large scale has not yet been considered a viable alternative in South Africa. The demand for land, however, and the need to protect our limited groundwater resources, suggests that alternative solutions to landfilling need to be investigated.

As a possible alternative energy resource, with job-creating potential for developing communities, the option of incineration as an alternative to landfilling must be given consideration in any future waste management planning strategies, with due regard to prevailing air-quality standards.

Table 11.4: Minimum requirements for landfill sites

LEGEND

- B^- No significant leachate produced
- B^+ Significant leachate produced
- R Requirement
- N Non-requirement
- F Special consideration to be given by departmental representative

	G GENERAL WASTE							
	C Communal landfill		**S** Small landfill		**M** Medium landfill		**L** Large landfill	
MINIMUM REQUIREMENTS / CATEGORY	B^-	B^+	B^-	B^+	B^-	B^+	B^-	B^+
Appoint responsible person	R	R	R	R	R	R	R	R
Classify proposed landfill site	R	R	R	R	R	R	R	R
Eliminate areas with fatal flaws	R	R	R	R	R	R	R	R
Identify candidate landfill sites	R	R	R	R	R	R	R	R
Buffer zone	200m	200m	500m	500m	F	F	F	F
Minimum unsaturated zone	2m	2m	2m	F	F	F	F	F
Rank sites as indicated	N	N	N	N	R	R	R	R
Site feasibility study	N	N	R	R	R	R	R	R
Site description	R	R	R	R	R	R	R	R
Complete permit application form	R	R	R	R	R	R	R	R
Preliminary geohydrological investigation	N	N	R	R	R	R	R	R
Preliminary environmental impact assessment	N	N	R	R	R	R	R	R
Identify critical factors	N	N	R	R	R	R	R	R
Assess critical factors	N	N	R	R	R	R	R	R
Confirm no fatal flaws	R	R	R	R	R	R	R	R
Consult interested and affected parties	R	R	R	R	R	R	R	R
Confirm best site	N	N	N	N	R	R	R	R
Compile and submit feasibility report (including maps)	N	N	R	R	R	R	R	R
Department of Water Affairs & Forestry confirmation of feasibility	N	N	R	R	R	R	R	R

Table 11.5: Minimum requirements for permitting a landfill site

LEGEND

- B^- No significant leachate produced
- B^+ Significant leachate produced
- R Requirement
- N Non-requirement
- F Special consideration to be given by departmental representative

	G GENERAL WASTE							
	C Communal landfill		**S** Small landfill		**M** Medium landfill		**L** Large landfill	
MINIMUM REQUIREMENTS / CATEGORY	B^-	B^+	B^-	B^+	B^-	B^+	B^-	B^+
Appoint responsible person	R	R	R	R	R	R	R	R
Confirm site classification	R	R	R	R	R	R	R	R
Landfill permit	R	R	R	R	R	R	R	R
Deal with department's regional office	R	R	R	R	R	R	R	R
Deal with department's head office	N	N	N	N	N	N	N	N
Permit application form	R	R	R	R	R	R	R	R
Site demarcated on a map	R	R	R	R	R	R	R	R
Site visit by state departments	F	F	F	F	R	R	R	R
Full permit application report	N	N	N	F	R	R	R	R
Feasibility study report	N	N	N	R	R	R	R	R
Geohydrological investigation report	N	N	R	R	R	R	R	R
Geological investigation report	N	N	R	R	R	R	R	R
Environmental impact assessment	N	N	N	R	R	R	R	R
Environmental impact control report	N	N	N	R	R	R	R	R
Landfill conceptual design	R	R	R	R	R	R	R	R
Landfill technical design	N	N	N	R	R	R	R	R
Approval of technical design by department	N	N	N	R	R	R	R	R
Development plan	R	R	R	R	R	R	R	R
Operation and maintenance plan	R	R	R	R	R	R	R	R
Closure/rehabilitation plan	R	R	R	R	R	R	R	R
End-use plan	N	N	R	R	R	R	R	R
Water quality monitoring plan	N	N	R	R	R	R	R	R
Amend title deed to prevent building development on closed landfill	N	N	R	R	R	R	R	R
Report change of ownership	R	R	R	R	R	R	R	R
Site inspection prior to commissioning	N	N	N	N	R	R	R	R

LEVELS OF SERVICE

For any collection service to be truly effective all waste must be removed completely from all storage and collection points. This, however, has many practical and cost implications. It is therefore important to establish a minimum standard that is acceptable and affordable by the community, based on the on-site storage facilities provided, the frequency of collection and the effectiveness of the service provided. All levels of service are therefore based on the premise that the local authority provides the necessary guidance and a minimum collection/removal service at least once a week rendering an acceptable level of cleanliness within the community.

There are many permutations in providing a waste collection service within the current South African scenario. Particular thought and attention should be given to the mix of levels of service and cost-effective options of servicing the Phase 1 disposal points (see Figure 11.6).

Level 1

This first level of service can be considered the minimum, where the residents are required to deliver their waste to specifically allocated communal disposal points (Phase 1 disposal). These disposal points could be specially designed masonry structures with embankments, or ready-made containers strategically placed. Should the option of fixed structures be selected, these would need to be cleared either by manual labour or mechanical means and the waste transferred to suitable vehicles for transport to the disposal site. The container option allows the use of either rear-end loader or special application vehicles that have the capability of lifting and transporting the containers in use.

Level 2

This level of service requires the local authority to provide for the collection of refuse from each household by the cheapest possible means (i.e.

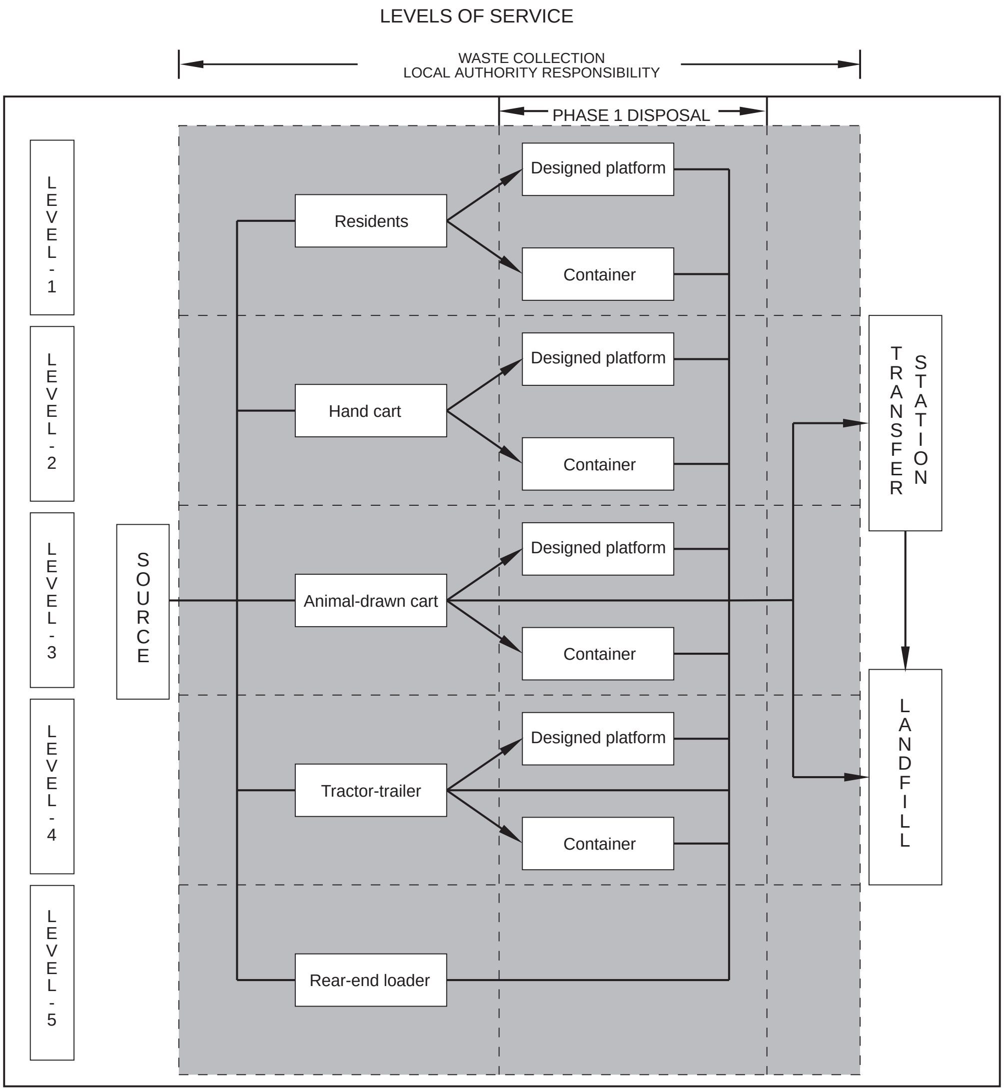


Figure 11.6: Levels of service

wheelbarrow or handcart). Refuse is collected from site or from the kerbside and transported to the nearest or designated communal disposal point.

Level 3

This level of service can be considered a natural progression from Level 2. The collection agent has the option of making use of animal-drawn carts or motorised tricycles, allowing for a larger area of responsibility. Where the landfill is considered to be an economically viable distance from the collection area, the need for Phase 1 disposal points may be eliminated, provided access via freeways is not required.

Level 4

This level of service can be considered a progression from Level 3, where the collection agent has the option of using mechanical means for transporting the collected refuse; depending on the terrain, this could be in the form of tractor-trailer or suitable light delivery vehicle. Again, should the landfill be considered an economically viable distance from the collection area, the need for Phase 1 disposal can be eliminated.

Level 5

This level of service can be considered the optimum means of collection in the majority of developed communities. The waste is collected from site or the kerbside by means of specially designed collection compactor vehicles and transported directly or via transfer station to point of final disposal.

GUIDELINES FOR IMPLEMENTATION OF LEVELS OF SERVICE

Levels of service 1 through 4 would as a rule only be implemented in conjunction with community-based programmes, or entrepreneurial development programmes in relatively small and developing communities. This, however, does not preclude their application in developed communities and elsewhere, or in combination with Level 5.

Table 11.6 provides some indication of the application, responsibility and conditions that need to be identified prior to implementation.

Quantity and composition

Proper and accurate estimation of the quantities and composition of the waste stream is an important aspect of the waste management system, as over- or under-estimation can result in either incorrect vehicle selection, shortened landfill life or increased costs.

The standard method of reporting waste generation is in terms of mass - this is necessary for proper vehicle selection. However, weight data are of limited value in designing landfills and selecting storage containers, which are identified by volumetric measurement. Volumetric measurement of the waste does, however, depend on how much the waste has been compacted and it's density.

There are two methods for determining waste-generation rates:

Load count analysis or weight volume analysis

- A specific area of collection is selected and the number of individual loads, corresponding vehicle volumes and characteristics, and the weight of each load are recorded over a specified period of time.

Materials balance analysis

- Identify a system boundary.
- Record all activities that cross and occur within the boundary, which affect the waste generation rate.
- Identify and record the rate of generation associated with these activities.
- Identify and record the composition of the waste generated within the boundary.

Collection-route balancing and planning

Once the quantity of waste needing collection is determined, it is important to plan collection routes to ensure a productive and economical service. This can be done by obtaining a map or plan of the area in order to identify the following:

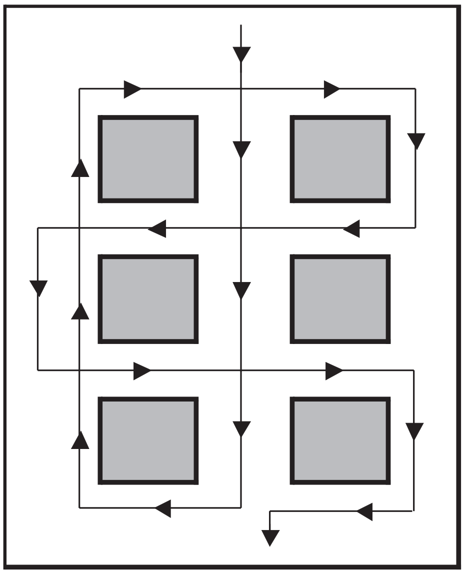
- all service points;
- all one way streets;
- any culs-de-sac; and
- areas that do not require a service.

Areas of daily collection should be compact and not fragmented and, where possible, natural boundaries should be used. Collection routes should be planned to maximise vehicle capacities. For the convenience of householders it is preferable to maintain a regular routine, to ensure their waste is ready for collection.

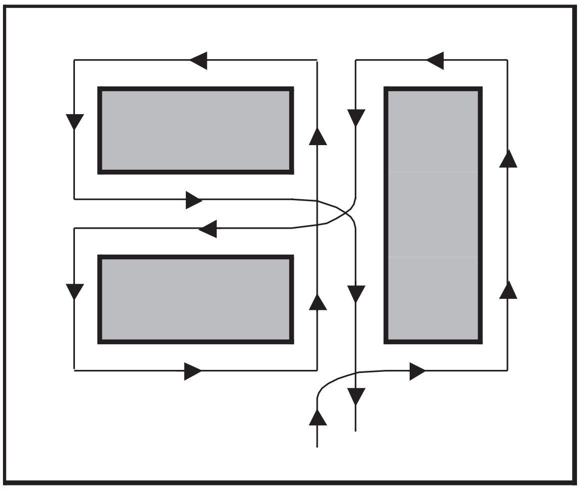
Table 11.6: Guide to responsibilities and conditions

LEVEL OF SERVICE	COMMUNITY	LOCAL AUTHORITY (and/or appointed agent)	CONDITIONS	REQUIREMENTS	
				DESCRIPTION	QUANTITY & TYPE
Level 1 Residents	Litter prevention Litter collection Delivery of waste to disposal point	Community liaison Collect waste from disposal point Maintenance of disposal point Environmental awareness programmes	No formal road infrastructure Maximum distance to disposal point ±200 m	Containers (minimum 1 per ±50 households)	Dependent on waste volumes Dependendent on frequency of service (Phase 1 disposal)
				Transport vehicles	Dependent on distance to landfill (see graphs)
Level 2 Hand carts	Place waste for collection on kerb-side Collect waste from site (where necessary) Litter prevention	Community liaison Collect waste from kerbside No formal road infrastructure Collect waste from disposal point Maintenance of disposal point Litter collection Environmental awareness programmes	No formal road infrastructure Maximum distance to disposal point ± 1 000 m	Hand carts	Dependent on community-based programme
				Containers (minimum 1 per ±50 households)	Dependent on waste volumes Dependent on frequency of service (Phase 1 disposal)
				Transport vehicles	Dependent on distance to landfill (see graphs) Dependent on frequency of service
Level 3 Animal-drawn carts; motorised tricycles	Place waste for collection on kerbside Litter prevention	Community liaison Collect waste from kerbside Collect waste from site (where necessary) Collect waste from disposal point Maintenance of disposal point Litter collection Environmental awareness programmes	No formal road infrastructure Limited road infrastructure Maximum distance to disposal point 3 000 m Allowable road ordinance conditions	Animal-drawn carts/Motorised tricycles	Dependent on community-based programme Dependent on waste volumes Dependent on frequency of service Dependent on access to landfill/transfer station
				Containers (minimum 1 per ±50 households)	Dependent on waste volumes Dependent on frequency of service (Phase 1 disposal)
				Transport vehicles	Dependent on distance to landfill (see graphs)
Level 4 Tractor-trailer combinations; light delivery vehicles	Place waste for collection on kerbside Litter prevention	Community liaison Collect waste from kerbside Collect waste from site (where necessary) Collect waste from disposal point Maintenance of disposal point Litter collection Environmental awareness programmes	Limited road infra-structure Maximum distance to disposal point ±10 km Allowable road ordinance conditions	Tractor-trailer combos/LDVs	Dependent on community-based programme Dependent on waste volumes Dependent on frequency of service Dependent on access to landfill/transfer station
				Containers (minimum 1 per ±50 households)	Dependent on waste volumes Dependent on frequency of service (Phase 1 disposal)
				Transport vehicles	Dependent on distance to landfill (see graphs)
Level 5 Rear-end loaders	Place waste for collection on kerbside Litter prevention	Collect waste from kerb-side Collect waste from site (where necessary) Litter collection Environmental awareness programmes	Allowable road ordinance conditions Proper road infra-structure	Transport vehicles	Dependent on waste volumes Dependent on frequency of service Dependent on distance to landfill (see graphs)

Detailed routing of collection vehicles can be planned for collection from both sides of the street or from one side only. Collection from one side only is preferable in business areas and areas with high traffic densities, to avoid conflict with traffic patterns and the safety of the workers. Figure 11.7 provides examples of collection-route alternatives.



ROUTE COLLECTION FROM TWO SIDES OF THE ROAD



ROUTE COLLECTION FROM ONE SIDE OF THE ROAD

Figure 11.7: Route collection alternatives

CONCLUSION

Waste management in the changing societies in South Africa is a dynamic process, in terms of both collection and disposal. It is the responsibility of the local authority to ensure the service is provided to its communities. Town and city engineers therefore need to be vigilant in identifying changes, and be innovative in making the service affordable, while meeting the standards expected by these communities.

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Chapter 12

Energy

Grid electricity

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SCOPE

These guidelines cover aspects that need to be considered for the optimum planning and designing of electricity supply systems for residential townships, including developing communities.

INTRODUCTION

This section provides electricity layout planners with the required background to effectively follow the correct steps to achieve optimal design. It describes the approach to "greenfield" projects with reference to the information, resources and documentation available to assist the planner/designer. Original content is kept to a minimum and there is effective referencing to established documents with explanatory notes where necessary. The guidelines are written with low technical content to maximise ease of reading and practical application.

NOTE: Reference materials are continually being updated to match developments in technology.

The use of typical analytical software tools is described and substantiated with practical examples.

Special attention is given to the methods that can be used to estimate After Diversity Maximum Demand (ADMD) figures and the ADMD vs consumer classification relationships.

Details regarding the method that should be followed when optimising an electrical layout and technology selection are given. Practical tips on service connection strategies and recommendations on the voltage drop calculation method are given.

(See Appendix A for lists of reference documents).

PLANNING OVERVIEW

Role of planning

Planning is vitally important to ensure the optimal application of technology and to achieve the required quality of supply. Planners have to consolidate and validate the information received from marketing surveys. Using the information at hand, a selection of technology (voltages and ratings) and its application (layout and configuration of network) must be decided upon. A strong feedback system is required to ensure that the as-built status of the network is in accordance with the plan. The planner is responsible for ensuring that future upgrades are properly planned and optimised.

Experience has shown that it is best to plan the development of an area in phases. The area will have an initial plan and a future envisaged masterplan. The implementation of the masterplan in phases is known as a "phased implementation plan". The terms "master plan", "initial plan" and "phased implementation plan" are used in the text.

Initial plan

It is unlikely to be economically feasible to implement the master plan immediately, and an initial plan that caters for expected network loading after five to seven years must be implemented first. The objective is to delay capital expenditure for as long as possible to reduce life cycle costs, while maintaining an acceptable quality of supply.

Master plan

The master plan layout refers to the long-range plan for the area (based on the expected loading after 20 years), using the optimised technology. The objective of master planning is to ensure upgradability and optimised long-term infrastructure development.

TERMS AND DEFINITIONS

A glossary of terms used in the planning and design of electrical distribution systems is given in the National Rationalised Specifications NRS 034-Part 0. A few terms have been extracted and adapted for ease of reading in the context of this chapter. Definitions are listed below in alphabetical order.

After diversity maximum demand (ADMD)

The simultaneous maximum demand of a group of *homogeneous* consumers, divided by the number of consumers, normally expressed in kVA.

Thus the ADMD of N consumers is:

$$\text{ADMD (N)} = \frac{\text{MD(N)}}{\text{N}}$$

This value generally decreases to an approximately constant value for 1 000 or more consumers and has therefore been chosen as a convenient reference value. (Practically no difference in ADMD exists between 100 and 1 000 consumers.)

ADMD with no mention of the number of consumers (N) is defined as that representing the ADMD of 1 000 consumers, i.e.

$$\text{ADMD} = \text{ADMD(1 000)}$$

For customers who have the potential to have a high or very high demand, an individual customer's maximum demand is generally approximately two to three times the ADMD for a group of similar customers. For customers with a limited potential demand, in the very low, low, or moderate consumption range, an individual customer's consumption is typically four to five times the ADMD for a group of similar customers.

DSP

A domestic supply point (consumer metering point).

Intermediate voltage (IV)

An AC medium voltage in the range 1 000 V to 3 300 V phase-to-phase.

Intermediate voltage distribution

Typically a distribution system operating at a nominal AC voltage of 1,9 kV phase-to-neutral, or 3,3 kV phase-to-phase.

Load factor (LF)

A ratio of the actual energy supplied (in kWh) over a period divided by the maximum demand in kW over that period, multiplied by the time period selected (i.e. actual energy supplied divided by potential energy supplied). It is always less or equal to unity.

$$LF = \frac{\text{Actual Energy}}{\text{MD (kVA)} \times \text{PF} \times t} \leq 1.0$$

$$\text{MD(kW)} = \text{MD (kVA)} \times \text{PF}$$

where PF is the power factor.

(See Figure 12.1.1.)

Low voltage (LV)

The range of AC voltages up to and including 1 000 V r.m.s. (see SABS 1019:1985 for a full definition).

Maximum demand (MD)

The highest averaged electrical demand for a specified period. Typically, 5 to 60 min and 30 min are normally used as these are close to the thermal constant of transformers and lines. (See Figure 12.1.1.)

Medium voltage (MV)

The range of AC voltages exceeding low voltage, up to and including 44 kV. (See SABS 1019:1985 for a full definition.)

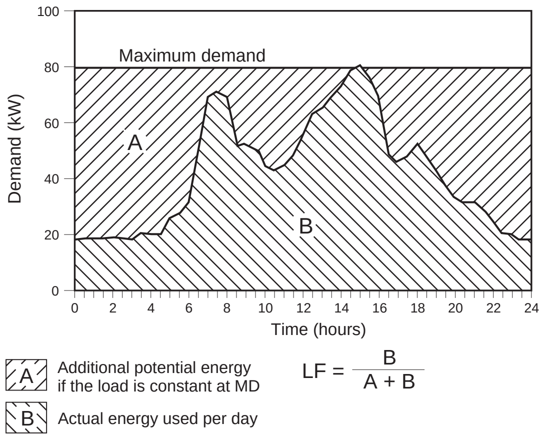

Figure 12.1.1: Typical daily demand curve

STATUTORY VOLTAGE LIMITS

The South African statutory voltage limits for the supply voltage to residential consumers have been summarised from Government Notice No R103 of 26 January 1996, which amended Regulation 9 of the Electricity Act (No 41 of 1987).

This notice stipulates that:

a) for nominal system voltages lower than 500 V, the supply voltage shall be the standard voltage;

b) in the case of a distribution system with a nominal system voltage lower than 500 V, the supply voltage shall not deviate from the standard voltage by more than 10%; and

c) the Board (i.e. The National Electricity Regulator) may on application permit other deviations from the stipulated supply voltage and frequency.

The statutory voltage limits imply that all new networks must conform to the standard voltage of 230 V ± 10% or 253 V maximum and 207 V minimum at the point of supply.

Typically, economic studies indicate the economic apportionment of voltage drop, to meet the statutory limits of 230 V ± 10%, is to aim for an approximately 8% voltage drop on the LV distributor, including service connections. This figure can be increased to 9% or even a bit higher, provided that the regulated busbar is electrically close to the electrification area and steps can be taken to make the LDC (load density classification) of the transformer respond only to the electrification area feeder's loading.

NOTES

- A further 5% voltage drop is permissible within the customer's premises (see SABS 0142).
- The assessment method for calculating the voltage from sample measurements is described in NRS 048-part 2.
- The regulated MV busbar must be controlled via LDC setting and electrification area distribution transformer tap positions (if fitted), to maintain at least 230 V during heavy load periods and not exceed 230 V + 10% during light load periods at the MV transformer's LV output.
- Care should be taken to limit the voltage drop in LV service cables near the end of LV distributors to less than 2% (using the appropriate undiversified ADMD), especially where long looped services supply a number of consumers.

ANALYSIS SOFTWARE

The availability of personal computers has led to the development of programs to assist with the analysis of networks. These packages are in some cases available from the market, but also from consultants and the network planning departments of electricity suppliers.

Three types of systems are available: ADMD modelling, low voltage simulation and electricity loss modelling.

ADMD modelling

An After Diversity Maximum Demand modelling approach is based on appliance usage information to calculate the ADMD, maximum demand, energy consumption and monthly electricity costs for any domestic community.

NOTE: This method should be used with extreme caution, because to obtain valid data on appliance availability and usage requires specialist market research.

Low voltage simulation packages

These are low voltage network simulation packages that integrate MV/LV transformer, LV distributor and service cables using per-phase analysis techniques. ADMD figures obtained from other systems are used as input.

Electricity loss modelling

This involves electrification loss-management systems that may be used to monitor system losses for implemented projects, or as design tools during the planning stage to help with network optimisation.

LOAD FORECASTING

Load forecasting impacts on the whole network plan, as well as capital expenditure. Residential township load forecasts focus on ADMD forecasting over a 15- to 20-year period, which reflects the economic life of the plant. ADMD is used for MV/LV transformer sizing and to calculate voltage drop over the LV feeders, service cables and MV/LV transformers.

Final vs initial ADMD

Two ADMD figures should be determined prior to any other analysis work or technology decisions:

- a seven-year (initial) ADMD figure is required to determine the first phase of the future master plan; and
- a master plan ADMD (final) for loads in 15 to 20 years' time is the major influencing parameter that determines the masterplan settlement layout.

Essential to the viability of the project is the use of a phased upgraded plan that progresses from the initial plan towards the master plan, using the respective ADMD values.

How to estimate ADMD

Three acceptable methods of determining the ADMD figures can be used:

Appliance model

This has an approach or design basis that models domestic appliance behaviour over the peak hour to estimate the appliance's contribution to the ADMD and the expected energy consumption per appliance per month.

This technique gives the planner the information required to forecast individual energy and demand requirement forecasting for the electrification area, and is the preferred method for ADMD determination.

NOTE: Recent experience with this method indicates that, for customers in the high and very high consumption classes, the consumption can be severely underestimated using this method.

Direct measurement

The ADMD may be determined by measuring the maximum demand of a representative existing suburb over the peak month (typically winter for Johannesburg). The suburb in question must ideally have been electrified for many years to yield the correct ADMD figure. If that suburb was provided with electricity only five years previously then a five-year ADMD figure will be determined, whereas the aim should be to establish a 20-year ADMD figure.

This method may therefore prove inappropriate for recently electrified suburbs.

Non-residential loads (such as for schools, hospitals and small industries) must be excluded from this measurement exercise to ensure homogeneity (only the peak time contribution of these loads must be subtracted from the overall measured ADMD, to take non-residential diversity into account).

Therefore,

$$MD_{\text{residential}} = MD_{\text{total}} - MD_{\text{non-residential}}$$

Energy load factor method

A further approach is that used where energy sales forecasts are available. The ADMD can be determined by estimating a load factor at suburb level, which is typically between 25% and 45% (if the individual DSP-level load factor is used then the ADMD will be undiversified, i.e. the demand for one DSP would be determined).

Example:

For a suburb the expected 20-year horizon for energy sales is estimated to be 2 400 units per annum, with expected sales of 1 440 units per annum after seven years.

The annual load factor for the township is estimated to be 26% for both time horizons.

Therefore,

$$ADMD_{\text{final}} = \frac{kWh}{LF \times h} = \frac{2\,400}{0{,}26 \times 8\,760} = 1{,}1 \text{ kVA}$$

$$ADMD_{\text{seven}} = \frac{kWh}{LF \times h} = \frac{1\,400}{0{,}26 \times 8\,760} = 0{,}6 \text{ kVA}$$

NOTE: The calculations are more appropriately done over annual periods than monthly periods, because there are significant differences in both consumption and load factor in different months.

CONSUMER CLASSIFICATION

Consumption classification

Consumers can be divided into classes according to annual consumption and appliance utilisation. NRS 034-1, Table 2, provides guidance for ADMD figures according to broad categories of consumption class. This table is given in Table 12.1.1 below, with annual load factors at suburb level (full diversity) added. These load factors have been derived from the NRS Load Research Project.

The results of investigations to correlate the annual energy consumption of consumers, the major classification characteristics of the consumers and the expected ADMD have been recorded in a report on the NRS Load Research Project, which is continuing. The results have been used to derive the planning parameters set out in the second edition of NRS 034-1.

Consumption classification can be derived from appropriate market research programmes and economic studies for each area to be electrified. This type of work is usually undertaken by consultants with the necessary marketing and engineering background.

$$ADMD\ (N) = \frac{MD_{\text{residential}}}{DSPs}$$

provided that DSPs > 100, to allow for full diversity.

If DSPs < 100 then adjust the ADMD as shown in Table 12.1.1.

Load density classification

Load density (kW/ km^2), which is a function of ADMD and stand size, is a very useful aid when selecting technology and calculating voltage drop. Table 12.1.2

Table 12.1.1: Consumption class

CONSUMPTION CLASS (SEE 4.3.3.2 OF NRS 034-1)	APPROX. FINAL LOADING AND DESIGN ADMD (kVA)	APPROX. ANNUAL LOAD FACTOR (%)	APPROX. kWh PER ANNUM
Very high	> 6	> 42*	> 22 000
High	3 to 6	35 to 42	9 200 to 22 000
Medium	1,5 to 3	31 to 35	4 100 to 9 200
Low	0,5 to 1,5	29 to 31	1 200 to 4 100
Very low	≤ 0,5	28 to 29	< 1 300

* In the very high consumption class, there might be demand-side management techniques, such as the application of ripple control that would influence the load factor.

Table 12.1.2: Domestic density classification

DOMESTIC DENSITY CLASSIFICATION	STAND SIZE (m^2)	AVERAGE LOAD DENSITY (kW/km^2)
URBAN:		(ADMD min = 0,5 kVA
		ADMD max = 4,5 kVA)
High density (HD)	<1 000	500 to 30 000
Medium density (MD)	1 000 to 4 000	300 to 5 000
Low density (LD)	4 000 to 20 000	100 to 1 500
RURAL:	>20 000	0,5 to 250

can be used to select a settlement's domestic density classification.

PLANNING PROCEDURE

A planning procedure based on an approach agreed upon between municipal engineers and Eskom is set out in NRS 034-1.

The recommended method for calculating voltage drops on LV distributors is to use the Herman beta algorithm described in NRS 034-1. The basis is statistical, and it can be conveniently implemented using spreadsheet software.

Available technologies

Currently available technologies for South African residential areas are:

- Conventional three-phase system with transformation from three-phase medium voltage to three-phase low voltage (e.g. 22/0,4 kV).
- Conventional single-phase system with transformation from medium voltage using two phases to single-phase low voltage (e.g. 22/0,23 kV).
- Intermediate voltage system using three-phase medium voltage source, stepping through two voltage transformations, first to an intermediate voltage and then to low voltage using either single- or three-phase alternatives (e.g. 22/3 kV, 3/0 kV, 4 kV for the three-phase option or 22/1 kV, 9/0 kV, 23 kV for the single-phase option).
- MV/LV maypole system for rural areas. This system usually employs smaller transformer sizes and the low voltage network consists only of service cables (e.g. 22/0,23 kV).
- Single wire earth return (SWER) system for very low loads and remote areas where the earth is used as a return load current path. The SWER line could, for example, be designed to be 22 kV between the ground and the single overhead wire (e.g. 22/0 kV, 23 kV).

Table 12.1.3 indicates the applications of these technologies for various domestic density classifications. For high-density urban applications the conventional three-phase system is the only appropriate one to use. For rural networks any system may be appropriate depending on the local conditions - even conventional three-phase systems may be used on a small scale where high-density pocket load areas are present. Economics will govern the decision-making process.

Table 12.1.3: High level preliminary technology selection

DOMESTIC CLASSIFICATION	TECHNOLOGY
URBAN high density (HD)	i
Medium density (MD)	i, ii
Low density (LD)	i, ii, iii
RURAL	i, ii, iii, iv, v

Planning steps

The following are the recommended steps for each of the selected technologies and their alternatives (test against a single representative sub-area to minimise optimisation time):

1. Select the domestic load density classification from Table 12.1.2.
2. The expected ADMD of the community can be determined for the five- to seven-year initial investment period and the final ADMD for the master plan using appropriate software and the information obtained from marketing surveys.
3. Do a high-level preliminary technology selection from Table 12.1.3. Various technologies would generally apply at this point.

4. Each selected technology constitutes a range of alternatives that must be tested to obtain the optimum arrangement of that specific technology. Therefore, choose the various conductors and transformer sizes to be evaluated to establish alternatives for each technology.

5. Determine the maximum feeder distance (intermediate voltage feeder, LV feeder or service connection, depending on the technology).

6. Determine the maximum number of feeders for each transformer size (intermediate voltage feeders, LV feeders or service connections, depending on the service cables - e.g. 22/0,23 kV).

7. Determine the total loss (kW) per configuration.

8. Determine the maximum number of DSPs per configuration.

9. Determine the cost for each configuration, including the cost of losses, plot these costs for each technology and choose a few of the lowest-cost options, keeping flexibility in mind for the masterplan.

   The various arrangements for each technology will produce an economic comparison or trade-off between transformer sizes and cable size. A typical result using this method is illustrated in Figure 12.1.2, which in this case produced an optimum LV feeder length of approximately 6.5X m. The graph is non-linear because, in general, larger cable sizes will be used for longer distances. The transformer and cable sizes that satisfy this minimum point should be used as the selected alternative for the technology.

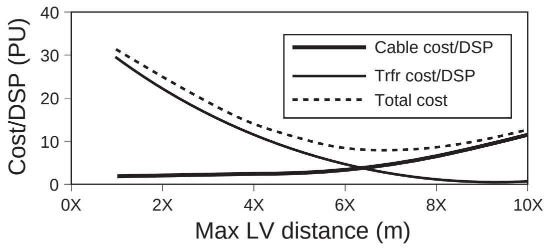

Figure 12.1.2: Economic trade-off

10. With the above information, identify the transformer supply areas from the layout for the chosen options, while maintaining flexibility for phased implementation.

11. Route the maximum number of LV feeders from the transformer positions, considering the information obtained from Step 6 for each option.

12. Calculate the voltage drop in a few typical and worst-case transformer supply areas. Change the feeder routes or lengths, if necessary, to meet voltage drop and conductor thermal constraints.

13. Complete the various design layouts to generate a bill of structures.

14. Use standard material costs to determine the masterplan cost and add the cost of losses using the guidelines in NRS 034-1.

15. Select the phased upgrade plan that defers the largest amount of the five- to seven-year capital cost, but will enable easy and inexpensive upgrading when the load materialises.

16. Life-cycle loss modelling forms an essential part of optimisation. It is therefore important that a simplified electrical layout model be subjected to loss calculations, to ensure optimal system loading. Do an energy-loss calculation, using appropriate software, and ensure close to equal load vs no-load transformer losses.

Area load densities

It is necessary to determine the load densities in kW/km^2 for the area under consideration. This value can be used to help with the preliminary selection of technology and sizing of transformers.

The first step is to determine the total load for the area by multiplying the final ADMD by the total number of DSPs.

Area load = ADMD x DSPs (kW)

Second, determine the total area to be supplied, excluding open spaces such as parks etc (in km^2).

<u>Example:</u>

ADMD = 1,2 kVA, DSPs = 1 500, Area = 1 km^2

Total load = 1,2 x 1 500 = 1 800 kW

Load density = 1 800 kW/km^2

Transformer supply areas

A maximum LV distributor length can be calculated using the stand size and ADMD. The area to be served by a single transformer is roughly described by a circle with radius equal to the distributor length.

Transformer supply area = πR^2

If R = 300 m then trfr. sup. area = 0,28 km^2

For a load density of 1 800 kW/km^2

Transformer load = 0,28 x 1 800 = 500 kW

A suitably rated transformer, taking account of its overload capability and the cyclic nature of the load, can be selected from the preferred sizes in the relevant specification (SABS 780:1979). For this example a transformer rated at 315 kVA would typically be suitable.

Master plan layout

The planner may start by drawing transformer supply circles on a site plan of the area. These circles have radii equal to the maximum LV distance that is allowed. Some overlap in the circles will naturally occur. Transformers may be positioned approximately in the centre of the circles. The entire LV layout can be established by working systematically from one end of the site to the other, making use of appropriate software to simulate voltage drop in the conductors. MV conductors may be modelled using load-flow software.

Practical considerations

After selecting the best technology option and arrangement for a particular settlement, the planner needs to do a complete electrical layout design and the following practical considerations are useful:

Initial layout

As discussed earlier, the initial phase is usually determined using the expected ADMD after seven years.

Installing only every second transformer planned for the master plan can reduce the number of transformers installed. The worst-case LV layout should be modelled to ensure acceptable voltage criteria, using the longer LV distributors (to the positions of those transformers removed). An alternative approach is to have reduced cable dimensions or single-phase arrangements for the initial layout, but then upgrading will be more costly.

MV operating criteria

The planner can determine the best MV system operation by specifying the location of normally open points, etc. MV voltage drop calculations are usually straightforward since, for the most part, only balanced three-phase alternatives are considered, with diversity allowed for as a scaling factor in repeated calculations.

It is important not to under-design the MV network, since this could be costly to replace at a future date. Use the maximum MV/LV transformer loads as point loads and do not diversify further. This will ensure sufficient MV line capacity to cater for future masterplan loading conditions.

Optimisation for urban application

Optimisation studies carried out in the electricity supply industry (ESI) used models that calculate, for high-density urban electrification application, the following information for various technologies:

- maximum distance per feeder;
- number of feeders to be supplied from a transformer;
- maximum transformer load;
- maximum number of consumers in transformer zone; and
- system losses.

From these results the total costs of all the configurations were determined and compared.

It was found that a three-phase LV design is optimal for high-density urban environments.

For stand sizes below 400 m^2, the use of three-phase LV 315 kVA transformers and a 35 mm^2 aerial bundled conductor (ABC) was in general the most cost-effective. For stand sizes between 400 m^2 and 1 000 m^2 the use of three-phase LV 200 kVA transformers and 70 mm^2 ABC was the most cost-effective alternative. In a medium density urban environment the best three-phase LV alternative was also found to be a 200 kVA transformer and 70 mm^2 ABC combination.

Single-phase LV and centre-tap LV designs were not only more expensive for this application, but exhibited the following disadvantages:

- limited reach;
- low efficiency;
- the centre-tap LV system will have imbalances in the primary system, although the secondary system is balanced; and
- the centre-tap LV transformer can only be loaded to 86% of its capacity.

Nevertheless, single-phase, dual-phase and SWER systems may often be more cost-effective alternatives for rural/low density applications.

Areas of cost saving

Although every system might potentially benefit from being uniquely designed, there is often a penalty in both design cost and time, and the cost of using non-standard items. Given resource constraints, the use of a limited selection of preferred sizes of standardised items should lead to overall cost savings. Most of the items that are used in electrification projects are the subject of NRS specifications that specify preferred sizes/types/ratings as agreed by electricity suppliers, through the Electricity Suppliers Liaison Committee (see Appendix A).

Practical examples:

- Depending on the ADMD, a first-time capital cost saving of between 5% and 25% can be achieved using 315 kVA transformers and 35 mm^2 ABC compared to designs using 100 kVA transformers and 35 mm^2 ABC.
- In low and very low consumption areas the use of 4 mm^2 service conductor rather than 10 mm^2 can result in further incremental savings.

Pre-payment systems

It was evident from the studies that the electricity dispenser (ED) and ready board (RB) are the two components that have a significant influence on electrification costs.

A clear strategic direction for domestic metering needs to be developed to optimise business functionality requirements, technology standardisation, and cost, especially for rural areas.

It is also clear that a phased-implementation approach will result in considerable savings due to the initial cost reduction and delay in capital expenditure.

For low and very low consumption areas, an alternative is the use of an "electricity control unit" (ECU) that combines the functions of a prepayment meter and a ready board into one housing. Typically this option will be suitable for consumers with a maximum supply requirement of 8 A.

LV VOLTAGE DROP CALCULATIONS

Voltage drop calculations seem straightforward and easy until phenomena such as diversity between consumer currents and unbalanced network loading become important facts to consider.

Recent research in South Africa has shown that the deterministic method used to calculate voltage drop, which was derived empirically and adapted from methods used in the UK, frequently overestimated the consumption and did not take account of all influencing factors. It also assumed a Gaussian distribution of customer loads, and the research has shown that the distributions are skewed, being better modelled as beta distributions.

Recommended method for voltage drop calculation

Herman beta methodology

The technique described in the second edition (1997) of NRS 034-1 takes account of important research that gave rise to the Herman beta methodology, which takes account of the statistical nature of residential loads and is sensitive to all influencing parameters. This is the recommended method for voltage drop calculations. It is intended that the factors applied will be reviewed and - if necessary - amended, as additional information is derived using data from the NRS load research project (see the section on "consumption classification" above).

Alternative method for voltage drop calculation

Monte Carlo simulation

A full Monte Carlo simulation implementation procedure can be used to simulate the actual network behaviour during peak periods. It is necessary to model the consumer load characteristics around domestic peak consumption periods to provide a load model for the simulation software.

However, the method is generally impractical for use in electrification project design, as it is time-consuming owing to the large number of simulations required. Its use is normally confined to applications in research.

Two methods of allocating consumer loads may be used: direct measurement of consumer currents and the use of statistical distribution functions.

Measurement of consumer currents

Metering equipment can be installed at randomly selected customer installations within a homogeneous group of customers. It is essential that the total group has the same classification to ensure similar energy-usage behaviour patterns. Current measurements at each installation, averaged over five-minute intervals, provide adequate resolution to capture the dynamic behaviour of individual customer currents.

Summation of all the customers' currents will produce a diversified profile over the selected measurement period. The peak day and peak five-minute period can then be selected. This summated current, divided by the number of DSPs gives a diversified maximum current per DSP. A kW value for ADMD is used, which is simply:

$$ADMD = I \times V_{nom}$$

with I the ADMD current value at time of peak, and V_{nom} the nominal LV phase voltage = 230 V.

SERVICE CABLE CONNECTION STRATEGIES

Service cable phase connections may have various configurations, and decisions regarding how many phases to take from each node or pole and how to arrange subsequent connections to minimise voltage drop must be made. One of the largest contributing factors to voltage drop in LV networks is the presence of neutral currents due to unbalanced loading conditions. Not much can be done about the behaviour of consumers and technical imbalance minimisation techniques must be provided.

Oscillating vs non-oscillating connections

It can be shown that an oscillating phase connection strategy is the best solution. The following explains the application of oscillating and non-oscillating phase connections. All examples use four service connections per node (pole or kiosk).

One phase per node

Planners often design electrification projects to cater for single-phase service cable take-off points via pole-top boxes or the equivalent. This is the simplest method of servicing individual households.

One phase per node, non-oscillating (RWB-RWB):

NODE	RED	WHITE	BLUE
1	4	0	0
2	0	4	0
3	0	0	4
4	4	0	0
5	0	4	0
6	0	0	4

One phase per node, oscillating (RWB-BWR):

NODE	RED	WHITE	BLUE
1	4	0	0
2	0	4	0
3	0	0	4
4	0	0	4
5	0	4	0
6	4	0	0

This connection strategy results in much improved balanced loading conditions and a significant reduction in neutral voltage drop.

Two phases per node

Oscillating phase connection taking two phases out per node produces the following connection arrangement:

RR-WW
BB-BB
WW-RR
RR-WW
BB-BB
WW-RR, etc.

Three phases per node

Oscillating phase connection taking three phases out per node produces the following connection arrangement:

RWBB
WRRW
BBWR
RWBB
WRRW
BBWR, etc.

NODE	RED	WHITE	BLUE
1	1	1	2
2	2	2	0
3	1	1	2
4	1	1	2
5	2	2	0
6	1	1	2

Using three phases per node decreases the voltage dropped as a result of the unbalanced condition by 10-20% over the two-phase per node case. Owing to this relatively small difference it is recommended that two phases be taken out per node and, in addition, that the oscillating phase connection strategy be used.

AREA LOSS MODEL

Planning proposals should include a full technical loss model with analysis results using forecasted energy-usage patterns and associated expected demands. Loss management should be incorporated as a basic requirement for all implemented electrification projects.

DOCUMENTATION REQUIREMENTS

Electrification projects should have a minimum set of documents, which include a planning proposal with the following:

- Maps of the area.
- List of evaluated technology options with technology cost comparisons as described in this document.
- Master plan layout and assumptions.
- Phased-implementation plan.
- MV loadflow calculation results showing system loading, losses and high/low voltages.
- MV/LV transformer and LV distributor plus service connection voltage-drop calculation results, using voltage profiles, LV distributor and MV/LV transformer loading. Results must include the master plan and the phased-implementation plan results.
- Loss evaluation of the system.
- Complete consultants brief if applicable.

See also Table 12.1.4.

Table 12.1.4: Summary of documentation required and examples

	MINIMUM INFORMATION	EXAMPLE
1.	**MASTER PLANNING**	
1.1	**Geographical load forecast**	
1.1.1	Domestic final energy consumption (kWh/month)	480 kWh/ month per consumer
1.1.2	Domestic final ADMD (kVA)	2,7 kVA
1.1.3	Domestic load factor (% for 1 000 consumers)	25%
1.1.4	Final number of households	7 000
1.1.5	Bulk loads and notified max demand (no and kW)	2 x 130 kVA and 6 x 70 kVA and 13 x 18 kVA
1.1.6	Bulk loads contributions to peak (no and kW)	2 x 70 kVA and 6 x 20 kVA and 13 x 6 kVA
1.1.7	Electrification area bulk supply demand (kW)	20 600 kW (assumed 7% peak demand losses)
1.1.8	Electrification area supply energy (kWh)	3 700 000 kWh/month (estimated)
1.1.9	Electrification size (km^2)	4,7 km^2
1.1.10	Average domestic sand size (m^2)	400 m^2
1.1.11	Load density of total area (kVA/km^2)	4 100
1.1.12	Load density of pocketed areas (kVA/km^2)	4 100 (no pocket loads in urban areas)
1.2	**Optimised technology selection**	
	Technology type	Conventional MV/LV
	General LV conductor choice (ABC 35 etc.)	35 mm^2 ABC
	Average LV conductor length	
	Max LV conductor length (m)	220 m
	Generally used transformer (kVA)	100/160 kVA ANSI CSP
	Reason for choice	Most economical for high density urban application
1.3	**Optimised voltage drop allocation**	
	Regulated % MV voltage during peak	103,5%
	% MV voltage at MV/LV transformer at peak	101% (lowest MV voltage)
2.	**PHASED-IMPLEMENTATION PLAN (in 5 to 7 years' time)**	
2.1	Domestic energy consumption (kWh/month)	260 kWh/month per consumer
2.2	Domestic ADMD (kVA)	1,8 kVA
2.3	Domestic load factor (% for 1 000 consumers)	20%
2.4	Number of households connected	4 000
2.5	Bulk loads and notified max demand (no and kW)	1 x 130 kVA and 2 x 70 kVA and 9 x 18 kVA
2.6	Bulk loads contributions to peak (no and kW)	1 x 40 kVA and 2 x 10 kVA and 9 x 4 kVA
2.7	Electrification area bulk supply demand (kW)	7 700 kW (assumed 5% peak demand losses)
2.8	Electrification area supply energy (kWh)	1 100 000 kWh/month (estimated)
2.9	Electrification size (km^2)	3,3 km^2
2.10	Load density of total area (kVA/km^2)	2 200
2.11	Load density of pocketed areas (kVA/km^2)	2 200 (no pocket loads)

FINANCIAL CALCULATIONS

Refer to NRS 034 part 1 for details on financial calculation methods.

APPENDIX A

REFERENCE STANDARDS, SPECIFICATIONS AND GUIDELINES

A.1 The NRS rationalised user specifications

The NRS specifications for application in the electricity supply industry are approved for use by the Electricity Suppliers Liaison Committee which comprises, inter alia, representation from the Association of Municipal Electricity Undertakings (AMEU) and Eskom. The committee provides considered choices of planning techniques, codes of practice, recommended practices and preferred equipment, as well as material types, sizes, and ratings, the widespread use of which should lead to overall cost benefits in the electrification drive.

NRS	TITLE
002:1990	Graphical symbols for electrical diagrams Amendment 1 - Index, architectural and reticulation symbols
003-1:1994 003-2:1993	Metal-clad switchgear, Part 1: 1 kV to 24 kV - Requirements for application in the ESI Part 2: Standardised panels
004:1991	Mini-substations, Part 1: up to 12 kV - Requirements for application in the ESI
005:1990	Distribution transformers: Preferred requirements for application in the ESI
006:1991	Switchgear - Metal-encl. ring main units, 1 kV to 24 kV
008:1991	Enclosures for cable terminations in air - Clearances for 7,2 kV to 36 kV
009 Series 009-2-2:1995 009-2-4:1997 009-4-1:1995 009-4-2:1993 009-6-1:1996	Electricity sales systems Part 2: Functional and performance requirements; Section 2: Credit dispensing units Section 4: Standard token translator Part 4: National electricity meter cards and associated numbering standards Section 1: National electricity meter cards Section 2: National electricity meter numbers Part 6: Interface standards; Section 1: Interface Credit Distribution Unit (CDU) to Standard Token Translator (STT)
013:1991	Electric power cables form 1 kV to 36 kV
016:1995	Code of Practice for earthing of low-voltage distribution systems
017 Series Part 1:1997 Part 2:1997	Single-phase cable for aerial service connections Split concentric cable Concentric cable
018 Series 018-1:1995 018-2:1995 018-3:1995 018-4:1996 018-5:1995	Fittings and connectors for LV overhead powerlines using aerial bundled conductors Part 1: Strain and suspension fittings for self-supporting conductors Part 2: Strain and suspension fittings for insulated supporting conductors Part 3: Strain and suspension fittings for bare supporting conductors Part 4: Strain and suspension fittings for service cables Part 5: Current-carrying connectors and joints

NRS	TITLE
020:1991	Aerial bundled conductor - Cable ties
022:1996	Stays and associated components (second edition)
025:1991	Photo-electric control units (PECUs) for lighting
027:1994	Electricity distribution - Distribution transformers - Completely self-protecting type
028:1993	Cable lugs and ferrules
029:1993	Outdoor-type current transformer
030:1993	Electromagnetic voltage transformers
031:1993	Alternating current disconnectors and earthing switches (above 1 000 V)
032:1993	Electricity distribution - Service distribution boxes - Pole-mounted (at 230 V)
033:1996	Guidelines for the application design, planning and construction of MV wooden pole overhead power lines above 1 kV and up to and including 22 kV
034 series 034-0:1998 034-1:1997 034-2-3:1997 034-3:1995	Guidelines for the provision of electrical distribution networks in residential areas Part 0:1998: Glossary of terms (in course of publication) Part 1: Planning and design of distribution systems Part 2-3: Preferred methods and materials for overhead lines Part 3: O/h distribution in low and moderate consumption areas
035:1994	Outdoor distribution cut-outs (drop-out fuses) - Pole-mounted type up to 22 kV
036 series 936-1:1994 036-2:1997 036-3:1997	Auto-reclosers and sectionalisers - Pole-mounted types Part 1: Programmable protection and remote control Part 2: Auto-reclosers with programmable protection Part 3: Sectionalisers
038-1:1997	Concrete poles, Part 1: Concrete poles for o/h distribution and reticulation systems
039:1995	Surge arresters - Application guide for distribution systems
040-1:1995 040-2:1994 040-3:1995	HV operating regulations, Part 1: Definitions of terms Part 2: Voltage colour coding Part 3: Model regulations
041:1995	Code of practice for overhead power lines for RSA
042:1996	Guide for the protection of electronic equipment against damaging transients

NRS	TITLE
043:1997	Code of practice for the joint use of a pole route for power and telecommunication lines
045:1997	Account card for electricity consumers
046:1997	Pole-mounted load switch disconnectors
048 series 048-1:1996 048-2:1996 048-3:1998	Quality of supply Part 1: Overview of implementation of standards and procedures Part 2: Minimum standards Part 3: Procedures for measurement and reporting

A.2 SABS standards

The following South African Bureau of Standards documents may assist in the planning, design and construction of electrical networks in residential areas.

SABS STANDARD	TITLE
97:1991	Electric cables - Impregnated-paper-insulated metal-sheathed cables for rated voltages from 3,3/3,3 kV up to 19/33 kV
152:1997	Low-voltage air-break switches, air-break disconnectors, air-break switch-disconnectors, and fuse-combination units
156:1977	Moulded-case circuit-breakers
171:1986	Surge arresters for low-voltage distribution systems
172:1994	Low-voltage fuses
177 (in 3 parts)	Insulators for overhead lines of nominal voltage exceeding 1 000 V
178:1970	Non-current-carrying line fittings for overhead power lines
753:1994	Pine poles, cross-arms and spacers for power distribution and telephone systems
754:1994	Eucalyptus poles, cross-arms and spacers for power distribution and telephone systems
780:1998	Distribution transformers
1029:1975	Miniature substations
1030:1975	Standard longitudinal miniature substations of ratings not exceeding 315 kVA
1180 (in 3 parts)	Electrical distribution boards
1268:1979	Polymeric or rubber-insulated, combined neutral/earth (CNE) cables with solid aluminium phase conductors and a concentric copper wire waveform combined neutral/earth conductor

SABS STANDARD	TITLE
1339:1992	Electric cables - Cross-linked polyethylene (XLPE)-insulated cables for voltages from 3,8/6,6 kV to 19/33 kV
1411	Materials of insulated electric cables and flexible cords
1418 (in 2 parts)	Aerial bundled conductor systems
1473	Low-voltage switchgear and controlgear assemblies
1507:1990	Electric cables with extruded solid dielectric insulation for fixed installations (300/500 V to 1 900/3 300 V)
1608:1994	Portable earthing gear for busbar systems and overhead lines
1619:1995	Small power distribution units (ready boards) for single-phase 230 V service connections
IEC 60076 (in 5 parts)	Power transformers
IEC 60099-1:1991	Surge arresters Part 1: Non-linear resistor type gapped surge arresters for AC systems
IEC 60099-4:1991	Surge arresters Part 4: Metal-oxide surge arresters without gaps for AC systems
IEC 60129:1984	Alternating current disconnectors and earthing switches
IEC 60265-1	High-voltage switches Part 1: High-voltage switches for rated voltages above 1 kV and less than 52 kV
IEC 60269-1 (in 6 parts and sections)	Low-voltage fuses
IEC 60282-1:1994	High-voltage fuses Part 1: Current-limiting fuses

Other forms of energy

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SCOPE

This section is intended to provide engineers, settlement planners and developers with information about energy sources and applications other than grid electricity. The scope of application includes both urban and rural settlements.

Important energy sources and applications in this context include

- energy efficient building design;
- the supply and use of hydrocarbon fuels (paraffin, LPG, coal, wood); and
- solar energy applications (in particular, solar water heating and the use of solar photovoltaic systems for off-grid electricity supply).

Guidelines are provided on the costs, applications, environmental and safety aspects, and relevant planning considerations associated with these energy sources and technologies.

Background information and references are also provided for other renewable energy technologies which are less widely used in South Africa, but which might be considered by planners in specific circumstances: wind turbines for electricity generation, small-scale hydropower, biogas digesters, extraction of landfill gas and solar-thermal electricity generation.

Some of the practices addressed in this section are still under development in South Africa and not yet well standardised. These topics are therefore addressed in a more descriptive and lengthy way. Improved concise guidelines should be possible in future revisions, as the practices become more widely established.

INTRODUCTION

The supply of grid electricity is the foremost energy infrastructure concern for planners, engineers and developers in South African urban settlements.

However, in lower-income urban communities - which include the majority of South Africa's urban population - grid electricity is only one part of energy supply and use. In practice, low-income households continue to use a variety of other energy sources, even when grid electricity is available. It is therefore useful for planners to consider these wider energy needs.

A second reason for considering energy alternatives to grid electricity is that, in dispersed or remote settlements (generally rural), grid electrification can be highly uneconomical. In this situation, decentralised "off-grid" sources of electricity may be preferable. These are nearly always more costly than a normal urban electricity supply, but nonetheless may be cheaper than grid extension and reticulation over long distances with low load densities. Again, electricity is only a part of the energy supply/use equation in such areas, and other fuel use must also be taken into account.

A third reason for examining energy alternatives arises from environmental concerns. Global environmental concerns about reducing pollution and greenhouse gas emissions have led to a greater awareness of the benefits of energy conservation and energy efficiency, as well as to a preference for using renewable energy sources where possible rather than fossil fuels which pollute the environment. Local environmental issues include serious concerns about

- the impact of smoky local indoor/outdoor environments on health;
- fires;
- contamination and poisoning; and
- land degradation caused by pollution from power stations and the over-exploitation of natural vegetation for energy purposes.

PLANNING CONSIDERATIONS

Most of the energy sources and applications treated in this section do not involve the provision of extensive physical infrastructure within settlements. Many of them involve economic and behavioural choices by individual consumers or households. Physical planning issues do arise, but other important consumer-oriented considerations include

- mechanisms to encourage the integration of thermally efficient design in low-cost housing developments;
- suitable distribution channels for commercial fuels, energy-related equipment, and appliances;
- finance and repayment mechanisms;
- social planning, consultation, education and awareness-building; and
- the application of technical, safety and environmental standards.

Relevant planning issues will be noted in each of the sub-sections below. In general, the following broad recommendations could be considered by planners:

- Understand local energy needs, consumption patterns and supply options.
- Recognise that electricity does not address all the energy needs of lower-income communities, and make provision for the optimal use of other energy sources.
- Ensure consumers are able to make well-informed choices about their energy options, through awareness and education campaigns. Include information about costs, health and safety.
- In physical planning, take account of the orientation and spacing of buildings for good use of passive solar design principles (alongside other factors affecting layout choices, such as security, variety, etc).
- Promote the energy-efficient and cost-saving design of buildings.
- Take account of the environmental impacts of different energy options.

ENERGY CONDITIONS IN SOUTH AFRICA

This section provides a brief overview of household energy consumption patterns in South Africa, the costs and availability of non-electric fuels and renewable energy options, and the distribution of renewable energy resources in South Africa.

Energy consumption patterns in South Africa

Overall, the residential sector accounts for approximately 20% of the energy consumption in SA. The percentage of households with access to grid electricity stood at about 66% in 1997-98 and is steadily increasing. However, in lower-income newly electrified settlements the average levels of electricity consumption per household are quite low - typically 50 to 150 kWh/month in the first few years after electrification (Davis 1995).

There are several reasons for this. Income constraints can make it difficult to afford higher electricity bills, and also difficult to acquire the more expensive energy-intensive electric appliances, such as stoves. Such households may therefore continue using existing cheaper appliances (such as primus stoves), cheaper fuels (such as coal for heating in winter) and non-commercial fuels like fuelwood and dung. Households without any electricity have, of course, to rely on such non-electric fuels and appliances.

"Fuel-switching" is a common phenomenon. It used to be thought that households in Southern Africa and similar developing countries would progressively switch from traditional fuels (e.g. wood) through transitional fuels (e.g. paraffin), then towards the partial use of electricity and finally to almost complete reliance on electricity. This may turn out to be a valid long-term trend, but in the medium term fuel-switching occurs in both directions, depending largely on economic circumstances and changes in prices. Multiple fuel use is likely to continue to be the norm for lower-income households in South Africa, both urban and rural, electrified and non-electrified.

The proportions of different fuels used tends to vary in different parts of the country, reflecting climatic differences (e.g. cold winters in the interior highveld) and local differences in prices (e.g. cheap coal in proximity to coalfields). Figure 12.2.1 illustrates this variation for households in four metropolitan centres.

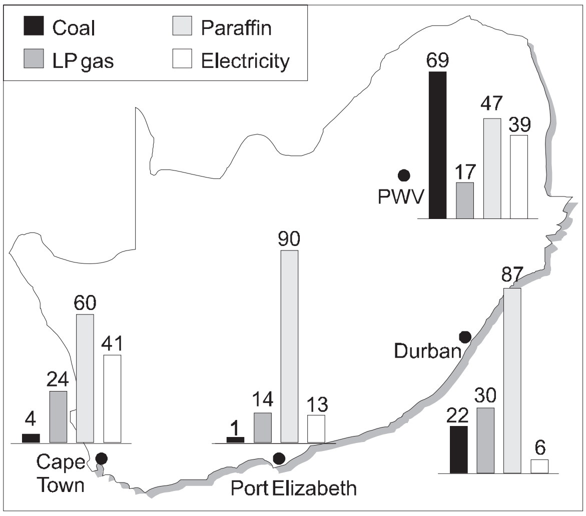

Figure 12.2.1: Percentage of households using different energy carriers, in four metropolitan areas
Note: Percentages total more than 100 %, indicating multiple fuel use by households.
Source: Williams 1993

Rural households make extensive use of wood and other biomass (crop residues, dung, etc) for cooking and heating. However, the use of wood also occurs in urban and peri-urban settlements. When wood can be collected "free" - although often involving considerable labour - it may be the most economically attractive or indeed the only energy option for low-income households. However, wood is also increasingly a commercial fuel, especially in areas of fuelwood scarcity.

Comparative energy costs

It is difficult to give accurate costs for the "useful" energy obtained from different fuel-appliance combinations, because the efficiency of the energy utilisation can vary. Table 12.2.1 provides estimated comparative costs for (a) the energy content and (b)

Table 12.2.1: Comparative consumer costs of different fuels for domestic cooking and heating (1997 prices)

	UNIT COST OF FUEL (UNITS AS SHOWN)		COST OF ENERGY CONTENT (CENTS/kWh)		ASSUMED EFFICIENCY OF UTILISATION	COST OF USEFUL ENERGY (CENTS/kWh)	
	GAUTENG	CAPE TOWN	GAUTENG	CAPE TOWN		GAUTENG	CAPE TOWN
COOKING							
Grid electricity	20 c/kWh	25 c/kWh	20	25	65%	31	38
Coal	26 c/kg	55 c/kg	4	8	15%	25	53
Paraffin	220 c/litre	168 c/litre	21	16	35%	61	47
LP gas	395 c/kg	321 c/kg	29	24	45%	64	52
Wood (commercial)	21 c/kg	45 c/kg	4	9	15%	28	60
COMBINED COOKING AND SPACE-HEAING							
Grid electricity	20 c/kWh	25 c/kWh	20	25	95%	21	26
Coal	26 c/kg	55 c/kg	4	8	45%	8	18
WATER HEATING							
Grid electricity	20 c/kWh	25 c/kWh	20	25	70%	29	36
LP gas	395 c/kg	321 c/kg	29	24	90%	32	26

Sources: Prices from Graham (1997). Efficiencies adapted from Fecher (1998), Gentles (1993), Lawrence et al (1993), Thorne 1995. Fuel prices can be higher from small traders and in rural areas.

the useful energy derived from different fuels, assuming the efficiencies shown, and based on typical 1997 prices for Gauteng and Cape Town.

Thermal energy services (cooking, heating, cooling) consume the largest amounts of household energy and are therefore a major concern for household choices about which fuels and appliances to use. As mentioned, the cost of the appliances themselves can be a deciding factor as well.

In general, poorer households spend a larger proportion of their income on energy services than more affluent households. They may also use less efficient appliances and pay more for the fuels they use, for example, by buying fuels like paraffin in small quantities. The use of candles for lighting is widespread in non-electrified households, but also in low-income electrified households. Households without an electricity supply often spend a large portion (e.g. one-third) of their overall energy budgets on dry-cell batteries for radios, etc (Hofmeyr 1994).

For lighting and media appliances (radio, TV, etc) small solar photovoltaic systems can provide a cost-effective alternative to paraffin lamps, candles and dry-cell or rechargeable batteries. The capital cost is higher, but life-cycle costs are lower for equivalent services. Table 12.2.2 gives examples of comparative lighting costs (using 1992 relative prices) for equivalent lighting levels.

Table 12.2.2: Comparative energy costs for 1 000 lumen-hours of lighting

Candles	R2,50
Paraffin lamp (wick)	R0,40
Pressurised paraffin lamp	R0,10
LP gas	R0,20
Solar home system	R0,05
Grid electricity	R0,002

Source: Cowan et al (1992)

For more detailed comparisons of fuel and appliance costs, variations in different parts of the country, life-cycle costs of fuel-appliance combinations and comparative costs of off-grid electricity options, see Energy & Development Group (1997), Graham (1997) and Cowan et al (1992).

THE DISTRIBUTION OF RENEWABLE ENERGY RESOURCES IN SOUTH AFRICA

Solar energy resources

Solar energy is generally abundant in South Africa, with a daily average of between 4 500 Wh (Durban) and 6 400 Wh (Upington) per square metre per day. These are values for solar irradiation measured on a horizontal surface (see Figure 12.2.2). For purposes of collecting solar energy, the solar collector is usually tilted to receive more energy throughout the year. Table 12.2.3 includes solar irradiation values for tilted surfaces. For many solar energy applications, a fixed-tilt angle is chosen which maximises the solar irradiation received during the "worst" solar month of the year, as shown in Table 12.2.3.

Solar radiation data for approximately 100 weather stations, and detailed data for the ten major South African measuring sites shown in Table 12.2.3, can be obtained in Eberhard (1990) and Cowan et al (1992); the latter source also includes software for calculating solar irradiation on north-facing tilted surfaces at different times of year, hour by hour.

For some purposes (e.g. in building design) it is useful to know the solar irradiation on vertical surfaces. Examples are provided in Table 12.2.4. Monthly values for all major weather stations are obtainable in Cowan et al (1992) and Eberhard (1990).

Table 12.2.4: Solar irradiation on vertical north-facing surfaces

MONTHLY MEAN IRRADIATION, IN Wh PER SQUARE METRE PER DAY		
Location	Maximum month	Minimum month
Bloemfontein	6 115 (Jul)	2 045 (Dec)
Cape Town	4 929 (Apr)	2 387 (Dec)
Durban	4 972 (Jun)	1 837 (Dec)
Pretoria	5 696 (Jun)	1 697 (Dec)

Source: Cowan et al (1992)

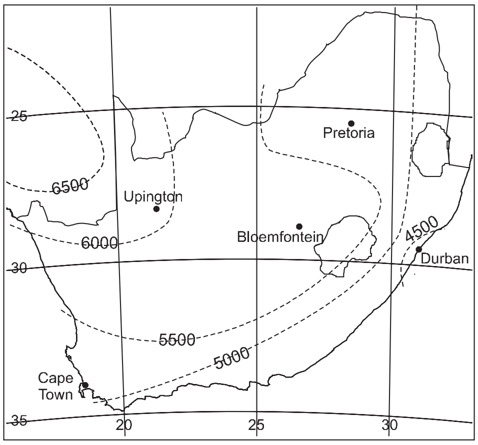

Figure 12.2.2: Annual mean solar irradiation on a horizontal surface (in Wh per square metre per day)

Source: Eberhard (1990)

Table 12.2.3: Solar irradiation values, from long-term measurements

LOCATION	MONTHLY MEAN SOLAR IRRADIATION, IN WH PER SQUARE METRE PER DAY				
	ON A HORIZONTAL SURFACE		ON A TILTED SURFACE (SEE NOTE)		TILT ANGLE (DEGREES)
	BEST MONTH	WORST MONTH	BEST MONTH	WORST MONTH	
Alexander Bay	8 364	3 555	6 732	6 164	45
Bloemfontein	8 096	3 747	7 031	6 318	35
Cape Town	7 956	2 390	6 275	4 702	60
Durban	5 740	3 033	5 488	4 759	35
Grootfontein	8 238	3 403	6 917	6 278	45
Nelspruit	6 045	3 919	5 765	5 108	20
Port Elizabeth	7 216	2 655	5 856	5 260	50
Pretoria	6 908	3 908	6 668	5 652	30
Roodeplaat	6 963	3 884	6 552	5 896	30
Upington	8 390	3 824	7 370	6 466	40

Source: Cowan et al (1992)
Note: A fixed north-facing tilt, in degrees from horizontal, which maximises solar irradiation for the worst month of the year.

Wind energy resources

Wind-speed measurements for South Africa are available from a large number of measuring stations across the country. However, the majority are agricultural stations and these wind measurements (usually at a height of 2 m and often in sheltered locations) are not always useful for judging wind-energy resources. Wind speeds can vary greatly over short distances in complex terrain. Wind-energy potential is highly sensitive to wind speeds, since doubling the wind speed increases the available power by a factor of eight. Average wind speeds of 6 ms^{-1} or more are favourable for electricity generation from wind, and wind speeds below 4,5 ms^{-1} are usually not viable, except for water pumping. Figure 12.2.3 provides an indication of wind-energy potential in different parts of South Africa. However, accurate site-specific wind assessments are essential to establish the viability of wind generation in a particular location. The most comprehensive source of analysed wind data, including frequency distributions and monthly variations, is Diab (1995).

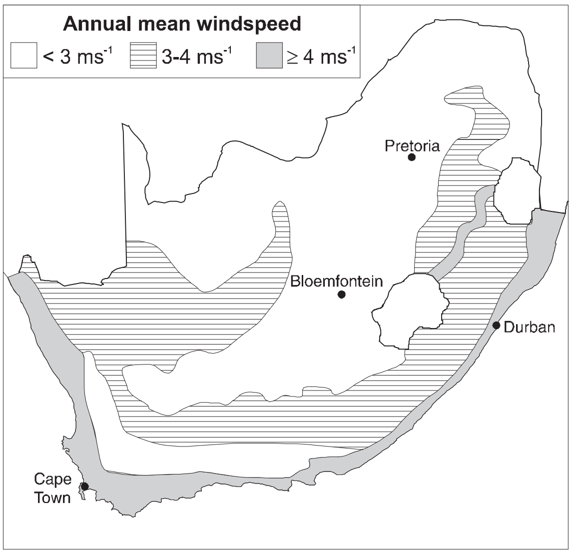

Figure 12.2.3: Annual mean wind speeds (normalised to 10 m height above ground, in ms^{-1})
Source: Adapted from Diab (1995)

Diab (1995) presents wind distribution statistics for 79 stations with hourly wind data. Of these, the following have wind data for at least two years, measured at a height greater than 2 m, and show annual mean wind speeds greater than 4 ms^{-1} (normalised to 10 m above ground level):

> 4,0 to 4,5 ms^{-1}	Alexander Bay, Beaufort West, Cape Town Airport, East London, Hermanus, Port Elizabeth, Stilbaai, Struisbaai, Vrede
> 4,5 to 5,0 ms^{-1}	Berg River, Wingfield
> 5,0 to 5,5 ms^{-1}	Gordons Bay
> 5,5 to 6,0 ms^{-1}	Buffeljagsbaai,Danger Point Die Gruis, Gansbaai
> 6,0 to 6,5 m^{s-1}	—
> 6,5 to 7,0 ms^{-1}	Waenhuiskrans

Other measurements of promising wind speeds, which either come from shorter periods of data or come from stations which were excluded from Diab's wind distribution statistics (and may not be fully reliable), are:

> 6,0 to 6,5 ms^{-1}	Bird Island, Klippepunt, Saldanha, Thyspunt
> 6,5 to 7,0 ms^{-1}	Cape Infanta, Hluhluwe
> 7,0 to 7,5 ms-1	Cape Recife, Hangklip
> 7,5 to 8,0 ms^{-1}	Seal Island

Hydropower resources

Hydro-power resources in South Africa are even more site-specific than wind. In general, the eastern slopes of the country from the Drakensberg down to the coast are more likely to offer the combination of sufficient water flow and gradient usually needed for viable small-scale hydro-electric installations (see later section on Small Hydropower).

ENERGY-EFFICIENT BUILDING DESIGN

Energy-efficient building design can help to reduce:

- excessive consumption of electricity or other fuels to maintain comfortable working, leisure or sleeping spaces;
- indoor/outdoor pollution; and
- health risks from a polluted or thermally uncomfortable environment.

This section discusses principles of building design, layout and user behaviour which can save energy, reduce costs and contribute to improved living environments. The focus is on information of interest to planners, designers, builders and building owners/occupants in residential areas.

Benefits of energy-efficient buildings

The energy consumption and expenditure required to maintain a comfortable (or in some cases simply bearable) indoor environment can be very significant, across all classes of housing and commercial building types. The potential for energy savings through improved energy-conscious building design can be as much as 70% in some cases. The "passive design" or "solar passive design" of buildings refers to construction/design techniques which

- make better use of natural energy flows (e.g. solar heating in the day, cooling at night);
- use building elements to insulate, capture, store or otherwise control energy flows; and
- reduce the need for "active" energy consumption/management.

Such techniques can be low-cost, long-lasting and offer the following potential benefits:

- lower energy bills for owners/occupants (especially in winter, when the energy bills of poorer households can rise significantly);
- reduced total electricity consumption - winter consumption levels are about 1 600 GWh/month higher than summer (Eskom, 1996);
- reduced electricity peak demand, associated with "cold snaps" (with resultant economic benefits both for electricity generation and the peak national/local distribution capacity required);
- less pollution (and lower health costs); and
- generally, a more pleasant and healthy environment in the home or workplace.

A particular South African health problem is respiratory disease caused by use of coal and wood for indoor cooking and space-heating (Terblanche et al 1993; Van Horen et al 1996). This could be partially alleviated by thermally efficient building design in low-income communities reliant on these fuels for heating.

Both the national electrification programme and the national housing programme present important drivers for improved thermal efficiency in housing:

- As the number of electrified houses increases, electric space-heating has an increasing impact on electricity consumption and winter peak demand (which will lead to more peaked and expensive generation, transmission and distribution).
- The national housing programme represents a unique opportunity to include low-cost thermal improvements which will reduce energy consumption for space-heating; if this opportunity is missed, it could result in long-term cost increases for residents, service providers and society at large (Simmonds 1997).

Guidelines for energy-conscious design

For specific climate zones and building types, it is possible to state relatively concise guidelines. However, the field is complex, and as Holm (1996) warns, over-simplification can lead to inappropriate application.

The approach adopted here is to discuss the principal elements of energy-conscious building design, and to give some indication of possible strategies. For more detailed information, Holm and Viljoen (1996) and Holm (1996) are useful resources. There are also numerous international texts in this field.

The Directorate: Settlement Policy of the Department of Housing (in association with other government departments) is in the process of drafting a set of guidelines for "environmentally sound low-cost housing".

Principal elements in managing indoor environments

Occupational comfort is related to temperature, air flow, lighting, humidity, the level of activity and clothing. The last two parameters are primarily under the control of occupants, and thus not discussed here. The other parameters are often actively managed in large commercial buildings, and to a lesser or greater extent in other buildings such as houses.

Energy-efficient or "passive" building design aims to reduce the active use of energy, and to achieve moderate variations around comfortable mean temperature (and other) conditions, using more economical means.

Figure 12.2.4 illustrates a cross-section through two rooms in a house, to show the main energy flows and parameters that influence the indoor environment.

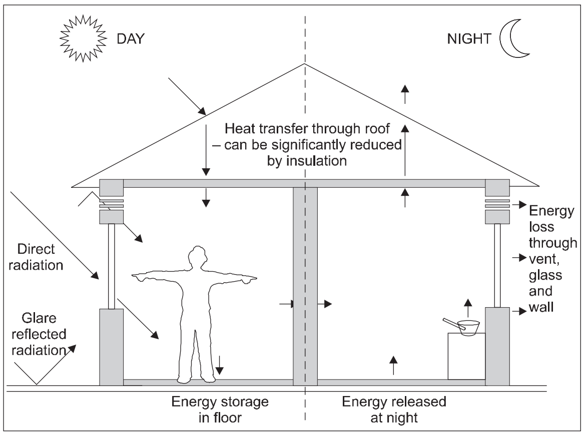

Figure 12.2.4: Principal energy flows in a home

Important elements of the energy flow diagram are:

- *energy source elements* - the sun shining through windows or heating external surfaces, human activity, appliances, heaters etc. in the home;
- *elements that remove energy from a home* - cooler air either flowing through a house or cooling external walls and roof, dark sky acting as a radiation sink at night, an air conditioner; and
- *energy stores within a home* - walls, floors, bodies of water within the home which can either absorb heat energy (helping to keep space temperatures lower) or give energy out (helping to warm a space).

Heat transfer is always from regions of higher temperature to regions of lower temperature. Three heat-transfer mechanisms are important:

- *conduction* through solid structures such as walls and roofing materials, which is highly dependent on the composition and thickness of the material (e.g. the conductivity of steel is a thousand times the conductivity of insulators like glass-fibre quilt);
- *convective* heat transfer, strongly related to air movement (which is enhanced by ventilation, or reduced through the use of multiple barriers or sealed cavities, as in a cavity wall); and
- *radiation*, proportional to the fourth power of the absolute temperature (Kelvin), and thus highly significant at elevated temperatures. Absorption of radiant energy also depends on the reflectivity of surfaces (highly polished surfaces and those painted with light colour paints are able to reflect most solar radiation while dark, rougher surfaces may absorb 95% or more).

Most heat transfer processes (e.g. from the air inside a room, through the wall to the outside) involve all three mechanisms. Tables listing heat transfer and thermal properties of common building materials are readily available (Wentzel et al 1981; Everett 1970; Burberry 1983; Quick II 1998).

Each of the principal building elements is discussed below, with attention given to opportunities for improvement.

Roof

Fifty to seventy percent of the heat loss from a home in winter can be through an uninsulated roof (Simmonds 1997). Furthermore an uninsulated roof will allow excessive heat gain during the day in summer. The addition of a ceiling can reduce space-heating energy bills by more than half and, even for homes with ceilings, the proper use of ceiling insulation can be a further cost-effective intervention in most climatic regions of South Africa (Temm International 1997; Van Wyk and Mathews 1996).

The following points need to be considered:

- Ceiling and insulation materials should be chosen with due consideration of other aspects of the building. There is little sense in adding thick, high-quality insulation to the ceiling of a draughty, thin-walled home. Cardboard or other low-cost insulators will achieve most of the benefits, at a fraction of the cost.
- Where buildings are otherwise thermally efficient, proper installation of thicker insulation with greater resistance to heat transfer is to be preferred.
- A gap of at least a few centimetres should be allowed between the top of the ceiling and the roof, as this will increase the insulation of the entire structure significantly, and allow ventilation.
- Materials of high reflectance and low emissivity (e.g. reflective aluminium-foil products) can also act as useful insulators, where radiation causes significant heat loss or unwanted gain (as in roof spaces).
- Where high thermal gradients occur across relatively thin insulation (e.g. cardboard ceilings) it is important to place a vapour barrier on the warm side of the insulation, to prevent condensation as warm, moist air comes into contact with cold exterior surfaces. Plastic sheeting or reflective aluminium foil is effective, if properly applied and sealed.
- Light colours tend to reflect more light. Heat gain from a roof is difficult to control in summer, so it is sensible to paint roofs a light colour.

Windows

Windows can be significant sources of both heat gain and loss in a home. Thermal performance will depend on size, orientation, shading, air tightness, material used for glazing, the use or absence of moveable insulation such as shutters or curtains, and on whether or not double glazing is used. Heat loss is primarily the result of convection, conduction and longwave radiation, all of which are significantly reduced by glass. Thus, provided the sun is shining, a window can cause heat gain even on a cold day.

- Windows that face east or west are more difficult to shade in summer (with greater potential for overheating). They should thus be kept relatively small, unless the user regulates the shading using moveable shades or other means discussed below.
- South-facing windows will tend to lose more energy than they gain; they are useful if the intention is to cool a space in summer, but will make it cold during winter. They can also be useful to collect diffuse light for work spaces.

CSIR (1997) provides an excellent paper-based method for calculating shadow angles at different orientations and times of the year, and provides helpful hints for controlling sunlight entry into buildings.

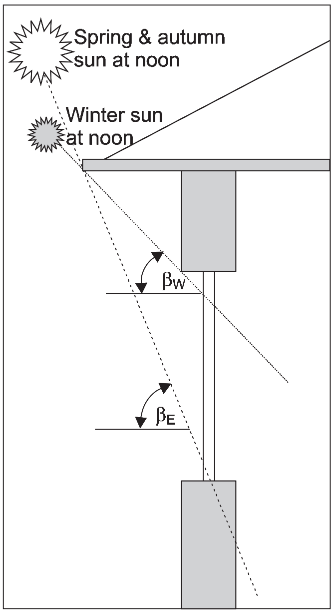

Figure 12.2.5: Shadow angles for north-facing windows
Note: These angles are for maximising winter gain ($\beta_W \geq 66.25^\circ$ - latitude), but avoiding overheating in summer ($\beta_E \leq 90^\circ$ - latitude). Eaves for orientations east or west of north will have to be longer, and it is difficult to avoid excessive solar gain.

Building occupants should manage windows, either opening or closing them to allow ventilation, or drawing curtains at night or in cold weather to minimise heat loss. To retain heat, curtains should be heavy, and/or windows should have close-fitting shutters. On the other hand, interior drapes will tend to absorb heat energy and release much of it into the building. Heat gain can most effectively be reduced using external shutters, louvres or highly reflective interior blinds. More expensive control measures, which are worthwhile in extreme climatic conditions, include double-glazing, adhesive films (which can be retrofitted) and low-emissivity glass.

Walls

Conductivity/insulation: Exterior walls should generally have good thermal insulation properties, to help keep heat out in summer and in during winter. Standard brick or block walls have reasonable thermal properties, especially if a cavity construction is used. Thin (single) brick, fibreboard and especially corrugated iron external walls are inefficient and cold in winter (or hot in summer) and should be insulated (but avoid using dangerously flammable materials).

Even low-cost insulation can reduce winter heat loss in corrugated iron dwellings by 50% (Weggelaar and Mathews 1998). If applied to 700 000 such dwellings in Gauteng, these authors estimate potential energy savings valued at R275-R450 million per year.

Thermal mass High thermal mass (the specific heat capacity x mass of an element) can have a moderating effect on the temperature inside a building. High thermal mass walls exposed to the sun or warm air during the day will warm up slowly (without overheating) and then re-radiate heat into the house overnight, helping to keep the home warm. This is particularly relevant in climates where day-night temperature variations are high (e.g. the highveld and interior regions of the country).

As with windows, north-facing massive uninsulated walls under eaves can be shaded in summer, but exposed to the sun in winter, providing a naturally moderated energy input to the home.

Wall colour: Light colours help to reflect heat. This can reduce overheating of thin, poorly insulated walls exposed to the summer sun.

The Trombe wall is a well-known specialised passive design technology which can be effectively used to warm a house in winter. An external glass pane shields the wall from wind, and significantly reduces convective and long-wave radiation losses, while allowing short-wave radiation from the sun to enter and warm the wall. These walls can also be used to enhance ventilation and facilitate cooling during hot weather, by placing vents that open to the outside near the top of the wall, and fitting temporary insulation on the inner surface. Air heated by the sun rises in the gap between wall and glass and leaves the house, drawing fresh, cooler air into the dwelling.

Other elements with high thermal mass

Floors, massive roof structures (earth or concrete) and specially designed elements such as water-filled structures all have high thermal capacity, and can be used to moderate indoor temperatures. In summer, a house can be fully opened at night, to lose as much heat as possible to the dark sky, and then closed during the day in an effort to keep the heat out and derive the benefit of the pre-cooled walls and floors.

On the other hand, if a building or room is used only for short periods, it may be appropriate to carpet the floors and use insulated walls of a low thermal mass, so that air-conditioning or heating will quickly get the room to the desired temperature without undue energy transfer to walls or floor.

In well-designed buildings, attention should be given to the balance of floor area, effective (exposed) thermal mass and north-facing window area. If window areas are too great for the house's thermal mass, then overheating can occur - even during winter days - and the house will cool down too rapidly at night. Windows that are too small will provide insufficient heat gain. Basic design guidelines are not given here, as appropriate ratios can vary significantly, depending on the thermal properties of different construction materials. Suffice it to say that optimum north-facing window areas are of the order of 10% or more of building floor area (Holm et al nd).

Ventilation

Adequate ventilation is essential to avoid overheating in summer, to avoid damp interiors and, perhaps most importantly, to remove pollutants released by indoor fires and coal stoves, where these are used. It is, however, important that such ventilation be controlled, and to a certain extent directed.

In hot weather, good cross-ventilation can be facilitated by windows or vents on both the leeward and the windward side of buildings. Exit windows should be high, to avoid warmer (lighter) air being trapped above them. Ceiling fans, and especially smaller fans directed as required, can significantly enhance user comfort in hot weather, at a much lower cost than air-conditioning.

Many South African households, however, are over-ventilated in winter, with poorly fitted doors, windows and ceilings. Again, potential savings are significant - in the order of 25% in some simulations performed by Van Wyk and Mathews (1996) on shack-type dwellings. "Weatherisation" - using caulking, weather strips, or even old cloths and cardboard - is a useful strategy.

Where smoky stoves or braziers are used, a direct chimney is better than extensive general ventilation to remove asphyxiating smoke and carbon monoxide. Space-heating efficiency is improved if there is also a specific air-supply vent for the stove/fire - otherwise the combustion draws cold air into the building.

Lighting

Daylight is cheap, provides good colour definition, and tends to have positive effects on people (compared to artificial lighting). Windows and skylights are well known, and work well. Where glare or direct sunlight is not welcome, diffusers or external shading devices can be advantageous. The success of shading devices is strongly dependent on orientation (see above). Excessive summer-heat gain from skylights can be a problem, as external shading is difficult to arrange. More specialised technologies such as solar tubes, and white or translucent reflectors/diffusers placed inside north-facing vertical glazed openings are sometimes useful and should be more widely used.

Humidity

The most important heat loss mechanism for humans is through the evaporation of sweat from the skin. If humidity levels are high, this evaporation is severely impaired, leading to discomfort at high ambient temperatures. Very low humidity leads to excessive dryness and is also undesirable. Reduced humidity levels are difficult to achieve using low-cost passive design features. Mechanical air-conditioning systems are generally used. However, in hot dry climates, air flow over ponds or through screens of wet porous material can be a highly effective method to increase humidity and cool the air.

The external environment

The texture and composition of surface material used around a building can have an important impact:

- Deciduous trees and vines on the sunny sides of a building will naturally adjust to the seasons, providing needed shade in summer, and allowing sunshine to reach the walls and windows in winter.

- Evergreen shrubs and bushes around the south-western to south-eastern borders can help to shelter houses from high winds (e.g. Western Cape) and provide a measure of insulation from cold winter skies in the highveld or interior.

- Low shrubs could be a disadvantage in humid, mild to hot climates, such as in coastal KwaZulu-Natal, where they will tend to obstruct the welcome movement of air. Trees with a high canopy are more desirable.

- Paving, especially on a northern or western aspect will reflect heat against walls and windows, and can add to heat build-up. If the paving is shaded by deciduous plants on a pergola in summer, however, the potential for enhanced heating can be turned to advantage in winter. Grassed areas will tend to be cooler.

- Pergolas and verandas can provide an important buffer zone around a building, especially with potted plants (their high thermal mass and active transpiration tend to cool the environment).

Site and building orientation

Buildings with a longer axis should preferably be orientated within 15°of an east-west line. This will tend to allow more windows, other openings such as doors, and increased wall area on the northern side to gain maximum benefit from the winter sun, while avoiding overheating in summer (see comments above regarding shade angles). Designers should locate living spaces on this northern side. Westerly orientations, in particular, should be avoided in hotter regions, as a baking afternoon sun can be very difficult to keep out. Northerly orientation can also facilitate the installation of solar water heaters (depending on roof design).

However, the effect of orientation is marginal for small, square structures that have small window areas. It should be stressed that other factors - such as access, security, variety, privacy/conviviality, slope and prevailing wind - should play a role in site layout and planning. Careful planning can still harness the benefits of passive solar design for a range of building orientations.

Solar access

Options for passive solar design can be severely restricted by adjacent buildings, especially on the northern side of a property. Site planners and designers should take careful cognisance of existing buildings, and future building possibilities, from the point of view of the current project as well as the needs of future developers on the southern side. In some countries solar rights are legislated, with shading of sites by adjacent buildings between 9 am and 3 pm prohibited (Holm and Viljoen 1996). Shared walls on the eastern and western sides can be an advantage, as they provide excellent insulation.

Site layout and development planning should be carried out with the above concerns regarding shading borne in mind. Road and stand plans should maximise the northern aspects, and avoid dense packing on south-facing slopes (where shading by adjacent buildings will be more severe).

Different climatic conditions

Most of the suggestions discussed above can be used to good effect to reduce the cost of energy services and enhance the built environment in all parts of South Africa. However, cost-effectiveness and appropriateness of different interventions is highly dependent on climate. The interior of the country generally has colder night-time and winter temperatures, and in some parts higher daytime summer temperatures. Thus insulation, winter solar gain and thermal mass are important. The Western Cape coastal regions tend to have a more temperate climate, and savings from passive design investments will not be as significant. In hot, humid areas such as coastal KwaZulu-Natal, summer ventilation and avoidance of heat gain (shaded walls, reflective insulation in roofs) are more important. Holm (1996) provides a detailed description of 13 climatic zones in Southern Africa, and provides design guidelines for each.

Economics and implementation

Changes to standard designs that bear relatively little cost - such as orientation of buildings towards the north, proper placement of windows for ventilation and summer shading - should be implemented wherever possible. Poor airtightness is frequently a result of poor quality control in construction, and can be improved by better training and quality control. Post-construction weatherisation can also be done low cost.

Cost-benefit estimates for more expensive options (installation of ceilings, insulation, etc) should consider three perspectives, namely those of:

- the householder;
- society at large; and
- the energy utility (especially the electricity-supply authorities).

Sometimes, the savings accruing to building owners may be insufficient to justify extra expense but, if the potential savings to the electricity-supply agency or society at large are also considered, a stronger motivation for an intervention can be made.

In general, ceilings and ceiling insulation will be cost-effective for all three parties in most parts of the country. The addition of low-cost insulation to single-skin corrugated-iron structures can also yield positive net benefits to householders. However, especially in coastal regions where ambient temperatures tend to be more stable, it may not be worthwhile from the householder's perspective to invest in ceiling insulation, or even a ceiling.

Estimates of the energy savings that can result from specific changes in design in specific climate zones can be derived from simulations or manual calculations - for example the CR method (Wentzel et al 1981). Computer programs are useful, some of which have been validated in South Africa (e.g. QUICK II 1998). Internationally developed and marketed programs such as DOE-2E, BLAST, and TRNSYS are also useful, provided that appropriate weather data are used. Such software can be sourced through enquiries addressed to the USA National Renewable Energy Laboratory, or similar institutions.

The viability of different interventions is also affected by the unit energy cost of the fuel being saved. Households using lower-cost heating fuels such as coal or wood will realise smaller financial savings than those using electricity. On the other hand, their health benefits may be greater.

Depending on a household's ability to set aside money for longer-term benefits (the effective "household discount rate" which balances capital investment against current demands on income), fuel savings from improved building design may not pay back the cost of the investment sufficiently quickly to be attractive. Poor people, in particular, can generally ill-afford capital expenditure, even if payback times are short.

Nevertheless, from the perspective of an electricity-supply utility (seeking to reduce peak demand), or the perspective of local or international society at large (seeking to reduce the impact of excessive energy consumption for space heating on health and the environment), it may be worthwhile to support such investments. This support can be in the form of

- direct subsidies;
- incentives offered for residential demand-side energy management; and
- careful structuring of finance packages (especially for new homes) to encourage people to choose better building designs which will save them money in the longer term.

A particular opportunity to support such investments, which is becoming more readily available, is access to international sources of finance, motivated by global environmental concerns.

Frequently, the pressure on housing developers is to deliver the maximum number of units possible, with minimum capital cost. In the absence of prescriptive legislation in South Africa to enforce construction methods that are more thermally efficient, this tends to result in householders and energy utilities being left with the long-term burden of high energy costs, and uncomfortable living environments. The development of social compacts between prospective homeowners, NGOs, planners and developers (and possibly with international funders) is an important element of building community-wide strategies to rationalise longer-term energy consumption.

HYDROCARBON FUELS

Paraffin

Paraffin is a liquid hydrocarbon fuel, which is produced by refining crude oil. It is very widely used in South Africa as a household fuel in non-electrified or recently electrified settlements.

Paraffin has a calorific value of 37,5 MJ/litre or 46 MJ/kg.

Utilisation and availability

Paraffin is mainly used in domestic applications for

- space heating;
- cooking;
- water heating;
- lighting; and
- refrigeration (less common).

Unlike liquified petroleum gas (LPG), paraffin is not suited to piped reticulation in buildings and is consequently not used in institutions or commercial enterprises.

Approximately 850 million litres of paraffin are used annually in South Africa (DME 1996).

Paraffin is widely available from small traders, garages and supermarkets in quantities determined by the customer's container size. The standard form of distribution is in 220 litre drums or 20-25 litre tins.

Appliances

A wide range of paraffin appliances is available. These may be pressurised (as in the case of a Primus stove or hurricane lamp) or non-pressurised, as in the case of a wick lamp. Pressurised appliances are more efficient and provide higher levels of output than non-pressurised units. Paraffin appliances tend to be portable and operate from a dedicated (or integral) fuel tank, which requires periodic refilling from a storage container.

Typical appliances include

- stoves - usually single-pot stoves;
- heaters - wick or pressurised;
- lamps - wick or pressurised hurricane-type; and
- fridges and freezers - usually wick-type units.

Appliances are widely available from general dealers, supermarkets and hardware stores.

Comparative costs

The retail price of paraffin is highly variable, due to the diversity in the distribution chain. Consequently, the retail price ranges from R2,50/litre upwards (October 1997 prices).

Hazards

The hazards associated with the use of paraffin include household fires, burns, asphyxiation and, most significantly, poisoning. Paraffin is poisonous in both liquid form and as a vapour. Up to 10 % of children in lower-income households suffer accidental poisoning (PASASA 1997). Children in the age group 1-4 years are most at risk.

Asphyxiation may occur as a result of the inhalation of poisonous paraffin fumes or due to the production of carbon dioxide and monoxide in confined or sealed rooms.

Safety measures

The high social costs of healthcare for paraffin-related injuries or poisoning have resulted in a concerted effort to provide practical measures and greater public awareness of safety in the use of paraffin.

The basic concerns include

- safe packaging and storage - dedicated containers with labels, safety caps, and a locked and ventilated storage area;

- safe utilisation - care of and correct use of appliances, safe handling by using a funnel, avoiding and cleaning spills, adequate ventilation of rooms; and

- health and safety awareness - information pamphlets, poison treatment cards, poison information centres.

The Paraffin Safety Association of South Africa, PASASA (tel: 0800 22 44 22) has developed a child-proof paraffin safety cap for commonly used containers. In addition, PASASA has developed paraffin safety labels for containers, safety posters and training packs for free distribution.

Planning considerations

As in the case of LPG (see below), the planning considerations for paraffin relate to the distribution of paraffin through a network of depots (or wholesalers) and small traders, and to general household fire and safety concerns.

Liquid petroleum gas (LPG)

Liquified petroleum gas (LPG) is a mixture of butane and propane gas stored under pressure, usually in steel cylinders. It is heavier than air, non-toxic and odourless. A smelling agent is added to aid users to detect leaks.

LPG is convenient as it has a high calorific value (energy content per kg) and is hence very portable, provides instantaneous heat, and is easy to ignite and clean-burning.

It is the most environmentally friendly of the commonly used fossil fuels because it is usually efficiently used and has low levels of harmful combustion products.

LPG has a calorific value of 49,7 MJ/kg or 13,8 kWh/kg, which is equivalent to 13,8 units of electricity.

Utilisation and availability

Approximately 280 million kilograms of LPG are consumed annually in South Africa (DME 1996).

Typical uses include

- space heating;
- cooking;
- water heating;
- refrigeration;
- lighting;
- workshop uses: brazing, soldering, welding; and
- school/technical laboratories

for use in

- domestic households;
- restaurants;
- hospitals;
- schools;
- small businesses; and
- recreation/leisure.

Appliances

LPG appliances include most of the commonly used electrical appliances other than electronic and/or motorised equipment. These include

- stoves/ovens;
- grills/braais;
- heaters (portable and fixed);
- instantaneous water heaters;
- lamps (portable and fixed);
- irons;
- refrigeration;
- welding plant; and
- bunsen burners.

The most commonly used LPG appliances are cooking, space heating and lighting appliances.

Appliances are designed to operate at an unregulated high pressure, such as in the CADAC (or similar) type of camping appliances, or at a lower pressure controlled by a regulator mounted on - or adjacent to - the gas storage cylinder. High-pressure and low-pressure appliances are not interchangeable.

Most households use gas appliances which operate off a small dedicated gas cylinder, rather than off a reticulated system of gas piping and fixed storage cylinders. Commonly used cylinder sizes are No 3 to No 10, and 9 kg cylinders (see Table 12.2.5).

Gas appliances must comply with SABS 1539:1991.

Comparative costs

The 1998 retail cost of LPG was R4,20 per kg (i.e. a unit energy cost of 8 c/MJ or 30 c/kWh).

The comparative effective costs per energy service are presented in Table 12.2.1.

Hazards

Typical hazards with LPG include:

- burns - common;
- explosions - quite uncommon; and
- asphyxiation due to displacement of air (by unburnt LPG in basements or on tanked floors or, more often, by carbon monoxide build-up in closed rooms).

Installation practices

LPG storage cylinders are supplied in a range of standard sizes in two categories: camping/hobby-type and household/industrial type.

The larger (9-48 kg) cylinders are designed to operate at 7 bar and are tested to 30 bar.

SABS 087:1975, Part 1, Code of practice for consumer LPG cylinder installation, is applicable to the storage of cylinders and gas piping in all cases when more than one cylinder, or a cylinder greater than 19 kg, is used on household premises.

In practice, it is recommended to use two cylinders, a duty and a standby cylinder, to ensure continuous service when the duty cylinder runs empty. It is also more convenient for households to manage two No 10 cylinders (4,5 kg) rather than one 9 kg cylinder.

Fixed gas installations must be installed by a registered gas installer (accredited installers can be checked with the LP Gas Association - refer to contact details below).

Guidelines for installation

The most important installation consideration is

Table 12.2.5: Standardised sizes of gas cylinders

CAMPING/ HOBBY HIGH PRESSURE APPLIANCES	HOUSEHOLD/INDUSTRIAL LOW-PRESSURE APPLIANCES	ENERGY CONTENT MJ	DIMENSIONS	
			DIAM. mm	HEIGHT mm
100 g		5		
200 g		10		
No 3		70		
No 7		160		
No 10		230		
	9 kg	450	305	406
	19 kg	950	305	762
	48 kg	2 400	381	1 143

the provision of good ventilation, preferably natural ventilation, in the storage area and the appliance area. Two airbricks or 150 x 150 mm weather louvres at ground level and high up on an exterior wall are recommended.

Storage of cylinders: outdoors, in a lockable and protected area, in an upright position.

Pipework: 10 mm Class 2 copper with brass compression fittings.

Regulators should comply with SABS 1237:1990.

Safety measures

The LP Gas Association (tel 011 886 9702 or 021 531 5785, PO Box 456, Pinegowrie, 2123) has developed a range of safety-information material and provides training courses for gas suppliers.

The basic safety issues include:

- Safe storage - safe handling and storage of cylinders.
- Safe utilisation - care of and correct use of appliances.
- Health and safety awareness - information pamphlets and training courses for suppliers.

Planning considerations

LPG distribution is provided via depots, which supply small traders or dealers such as general dealers or hardware stores.

Small traders typically supply approximately 2 000 kg/month which corresponds to settlements of between 500-1 000 households or 2 000-10 000 people, depending on the average monthly consumption and household sizes.

An average coverage per small trader could be 3 000 people.

An LPG depot is capable of distributing cylinders to 10-15 small traders.

Coal

Household use of coal

Coal (bituminous or ordinary coal) is one of the lowest-cost fuels available to households for space heating, and ranks on a par with most other fuels when used for cooking and water-heating (see Table 12.2.1). As a result it is widely used in Gauteng, parts of KwaZulu-Natal and in rural and urban settlements close to coal mining districts (refer to Figure 12.2.1). About one million households use coal in these areas (5 to 6 million people). Coal use is less common in other areas, partly because of higher prices due to transportation costs.

Typical appliances include

- cast iron "coal stoves" with closed combustion chambers exiting to a chimney (such as the Dover stove); and
- open and closed braziers, with the Mbaula being a common example; these do not have a chimney and are normally ignited outside, before being brought into the home.

Environmental and health issues

The coal generally used by householders is of relatively low quality (C or D grade), as this is significantly cheaper than higher grade coals. It has an energy content greater than 21 MJ/kg, an ash content of 15% to 50%, and releases noxious volatiles on combustion. The sulphur content is relatively low (1-2%). When burnt relatively inefficiently (as in a stove or Mbaula), the dangerous pollutants include particulates and noxious gases such as sulphur dioxide. Aside from general concerns regarding unpleasant smog and grime on buildings, the indoor and outdoor pollution levels in communities that use coal are high enough to markedly increase the risks of respiratory tract illnesses, with the use of coal in the home increasing risk by a factor of nine in one study (Van Horen et al 1996). If homes are under-ventilated, there is also a risk of asphyxiation from carbon monoxide poisoning. The direct health risks attached to household coal combustion thus have high personal and societal costs.

Efforts to reduce pollution

Strategies to reduce indoor and outdoor pollution from residential coal consumption include

- encouraging the substitution of coal use by other fuels which are cleaner, but require different appliances (LPG, paraffin, electricity);
- the provision of other fuels which can replace coal in existing coal-burning appliances (low-smoke coal, wood, briquettes);
- legislation to reduce allowable smoke emissions and/or prohibit coal use;
- improvement of stove combustion efficiency;
- education on the need to ignite Mbaulas outside the home, only bringing them inside

once smoke production has reduced;

- education regarding proper lighting methods (starting a fire on top of a pile of coal rather than underneath can reduce the amount of smoke produced);
- education regarding proper ventilation;
- substitution of open braziers and Mbaulas by stoves with chimneys (to remove smoke from indoor environment); and
- improving the thermal performance of houses to reduce the requirement for space heating (refer to the energy-efficient building design section above).

Although many of the interventions listed above fall primarily within the responsibility of coal users themselves, the effects of pollution affect society as a whole, and it is therefore important for planners, local authorities and national agencies to be directly involved. Some of the above strategies are discussed briefly below.

Substitution of coal with different "clean" fuels (e.g. electricity, gas). Both electrification and the provision of gas have the potential to reduce coal combustion in homes, reducing local pollution effects to a minimum. Experience to date, however, indicates that coal stoves continue to be used even after settlement electrification (Van Horen et al 1996). Costs to the householder remain an important barrier to fuel substitution. Coal can be the cheapest fuel for thermal applications (especially space heating - Table 12.2.1). Furthermore, replacement of the multi-purpose coal stove can require the purchase of three appliances: a stove, a water heater and one or more space heaters. Strategies to increase the rate of transition to electricity include

- the subsidisation of new-appliance costs (or improving access to credit for appliance purchase); and
- the subsidisation of electricity tariffs for low-income households (currently being mooted by the National Electricity Regulator).

Similar strategies can be used to promote gas use.

Substitution of coal with low-smoke coal or other similar solid fuels. A number of "low-smoke coal" fuels have been developed. These offer the potential of easier substitution in the market place: householders would not have to purchase new appliances or change their cooking and space-heating social/behavioural practices. These fuels can also be marketed through existing coal dealer networks, reducing disruptive effects on employment, etc.

However, low-smoke coals are more expensive than ordinary coal, with production costs three or more times the pithead price of bituminous (ordinary) coal. Even if one assumes savings in distribution costs for locally produced fuels, low-smoke fuels retail prices are from 1,7 to 10 times the retail price of coal (on a per-unit energy basis). Since they have relatively few advantages to the customer (apart from low smoke emission), it is clear that some form of market intervention would be necessary to achieve more widespread use. For a discussion of fuel types and market intervention options see Van Horen et al (1995).

Three products have recently been tested in a large-scale experiment:

- Two brands of devolatilised discard coal (produced by heating discard coal under controlled conditions).
- Compressed paper with a binder. User and merchant reaction was mixed. The devolatilised coal fuels, in particular, were difficult to light, initially blocked grates because they had too high a percentage of fines, and did not have good heat retention. The paper-based fuel was much easier to use, although it also did not have good heat-retention properties. There are thus a number of problems still to be overcome, before it will be possible to actively market a product that can compete directly with coal (Asamoah et al 1998).

Legislation of smoke-control zones

Part III of the Atmospheric Pollution Prevention Act (No 45 of 1965) makes provision for smoke control zones in which the sale or combustion of coal for domestic use can be controlled. Enforcement is usually through an education process, with warnings or "abatement" notices being served prior to prosecution for an offence in terms of the Act. Although effective in wealthier communities, where alternatives to coal are affordable, the Act cannot be expected to be observed in poorer communities where there is little economic alternative to coal. It is thus necessary to develop and implement other viable strategies before considering enforcing the observance of a legislated smoke-control zone.

Education and awareness

National or regional strategies can be effective only in as much as they are taken on board by communities and individuals. It is therefore important to engage in debate and education programmes, and generally raise awareness around coal pollution and abatement strategies.

Biomass

Biomass is extensively used for cooking in South Africa, with an estimated 16 million people relying predominantly on wood for space heating and cooking needs (Van Horen et al 1996). People tend to use open or sheltered fires, usually with a few stones or bricks to support pots or a grid. Varieties of "improved wood stoves" are available, but have not been extensively used or marketed in South Africa. Mean annual per capita consumption of biomass varies depending on the availability of wood, and the degree of substitution with other fuels such as paraffin, gas or even electricity. Typical values are from 350-700 kg per capita per year. Wood is usually collected at low or zero direct financial cost to the household. However, collection times are long (often three or more hours per trip), with multiple trips required per week. Commercialisation of the fuelwood market in rural areas is increasing. Prices are of the order of 20c to 50c per kg (1997-98), depending on availability and quantities purchased.

Concerns regarding fuelwood use

Wood is in many ways an attractive fuel for residential energy supply, as it is low cost (see Table 12.2.1), popular, familiar to people, and the resource is generally well distributed. If harvested on a sustainable basis the net greenhouse gas emissions are zero, and the resource is renewable. However, there are health and broader environmental concerns which should be addressed.

Health related concerns include the following:

- Burning wood brings increased risk of respiratory tract infection and eye diseases as a result of exposure to smoke in poorly ventilated houses. Studies have recorded exposures to total suspended particles from two to eight times recognised international standards (Van Horen et al 1996).
- Even if ventilation is provided, cooks may be so close to the fire that smoke exposure is still high.
- In denser settlements, outdoor air pollution from woodsmoke may also be a problem.
- The risk of burns, especially to children around open fires, is increased.
- Spinal injury and other personal-safety risks are associated with wood-collection trips.

Supply sustainability concerns include the decreasing availability of wood fuel as a result of

- unsustainable harvesting pressure and practices (environmental denudation); and
- competing land use claims for agriculture and housing development.

Biomass strategies

Key strategies for improvement of health and safety in the household environment include the following:

- Substitution of fuelwood with cleaner fuels (electricity, gas, paraffin). However, where wood remains significantly cheaper than alternative fuels, such strategies are expected to have limited impact.
- Greater use of improved stoves with chimneys. Such stoves should remove all smoke from the immediate cooking environment and also have good combustion efficiency to reduce total emissions. "Cooking efficiency", which is related to combustion efficiency, heat transfer efficiency, and the ability to control the rate of combustion are also important. A key difficulty is the establishment of a sustainable market that can deliver products of adequate standard.

Concerns regarding sustainability of supply can be partly addressed by reducing consumption. However, what is more important is the achievement of an integrated development and resource utilisation framework which takes into account land-use priorities, institutional and private ownership constraints, and biomass energy requirements. Measures directly related to biomass supply include the following:

- Redistribution of wood from areas of surplus to areas of need, preferably using commercially sustainable mechanisms. Sources of potential surplus include commercial farms, commercial forests, water-catchment areas infested with alien vegetation, and farm and game management areas suffering from bush encroachment.
- Encouragement of sustainable harvesting practices. These are often already practised by communities and include collection of dead wood only, cutting of selected branches to allow coppicing to occur, and careful selection of trees to be felled on the basis of size, position and species. Where coppicing does occur it is often useful to prune new shoots selectively to achieve more rapid re-growth of the remaining shoots.
- Development of tree planting and management strategies can enhance the supply

of wood for fuel and other purposes (construction, fencing, carving, etc). Options include woodlots, agroforestry and social forestry.

Woodlots require the establishment and management of small- to medium-sized plantations to produce fuel and other forest products. They have met with mixed success, as management and ownership structures are difficult to establish and maintain in the long term. However, where suitable local authority management structures are in place, they can be effective.

Agroforestry involves the planting of trees in association with crops. This can have benefits for crop production (shade, windbreaks, nitrogen enrichment).

Social forestry is a broad term, referring to community-based resource management involving the planting and management of trees in woodlots, as part of agroforestry, as part of land-reclamation processes, or in greening projects. Greening projects are perhaps most important in terms of settlement development and planning, as they can result in more pleasing and comfortable living spaces and greater access to fruit, wood and other products, depending on species and planting practice (Gandar 1994).

The Directorate of Forestry (Department of Water Affairs & Forestry) is the national entity responsible for overseeing many of the above strategies.

Town gas

Town gas, also called "producer gas", is the term used to describe reticulated gas, which is supplied in older parts of South African cities, such as Woodstock and Observatory in Cape Town, Melville in Johannesburg and parts of Durban and Port Elizabeth.

The gas is usually produced from coal, using a coal gasifier, or as a by-product of the chemical industry. Its calorific value varies, depending on the source, method of manufacture and supply pressure, ranging from 13,3 to 34,0 MJ/m^3.

The gas is reticulated in pipes (or gas mains) in the streets and supplied onto each erf via a gas meter, as for water or electricity.

Town gas is currently not considered for any new settlements, although this may change if the Pande natural gas reserves in Mozambique are developed and marketed in Mpumalanga and Gauteng.

Appliances

Built-in appliances such as instantaneous water heaters, cooking stoves and "gas fires" (space heaters) are commonly supplied by town gas.

In general, town-gas appliances operate at a lower pressure than bottled LPG appliances. Consequently, although most gas appliances can operate off either type of supply, they would require modifications to the jets or burners to adapt to the difference in pressure.

Safety

The safety considerations for LPG appliances apply equally to town gas appliances. However, the protection and marking of underground gas pipes also requires attention, to avoid accidental damage and leaks.

Planning considerations

No planning for town gas is required until town gas becomes an available option for new settlements.

SOLAR ENERGY APPLICATIONS IN SOUTH AFRICA

Provision of energy to sites and buildings usually requires the development of infrastructure to deliver the energy carrier (electricity, town gas, LPG, paraffin, etc) to properties. Solar energy is, however, generally available to householders. The following sections discuss the methods and mechanisms to facilitate the use of freely available solar energy for services such as water heating and electricity provision.

The Solar Energy Society of Southern Africa (SESSA) may be contacted for information at PO Box 152, La Montagne 0184, tel (012) 804 3435, fax (012) 804 5691.

Solar photovoltaic electricity supply

Photovoltaic (PV) modules are semiconductor devices that convert the radiant energy of the sun into a direct current (DC) electrical source. These can be coupled with energy storage devices such as batteries and power management equipment, to form an integrated system capable of delivering electricity for wide range of application. Simple systems usually have DC outputs to DC appliances such as lights, television, radio or hi-fi. Larger systems often incorporate a DC-AC inverter to power AC loads. PV systems are modular, have low enviromental impact, are quiet and have a long life (15 year warranties on the PV modules are common). Unit costs of energy are relatively high (R3,60 to R5,00 per kWh, in 1997 rands). However, PV systems can be installed at the point of need, thus significantly reducing the capital expenditure required for grid extension and even local distribution.

Typical applications of PV systems

Typical applications are listed below, with some indication of the number of installations in South Africa as at September 1998.[1]

- power for telecommunications (between 60 000 and 100 000 systems, or more than 2 MW_p installed);
- rural health-centre service provision (communications, vaccine refrigeration, lighting), with more than 180 such systems installed;
- school lighting and educational equipment power supply (more than 1 200 systems installed);
- domestic lighting, media and other high-quality energy supply needs:
  - individual household systems, often called Solar Home Systems (SHS), with an estimated 50 000 to 60 000 systems installed
  - battery-charging systems (shared by a number of users), a few pilot projects in operation
  - mini-grid reticulation systems (very little SA experience to date);
- water pumping (thousands of systems have been installed - see solar water pumping section below); and
- galvanic protection of structures against corrosion.

Potentially important applications of PV systems which have not yet been implemented on a significant scale in South Africa include

- street lighting in unelectrified communities or along roads not served by nearby sub-stations;
- grid-connected distributed generation systems - for example to provide reinforcement for daytime peak loads at the end of a long line which has insufficient capacity, or for back-up power; and
- building-integrated systems in which the PV modules also serve as facade or roof covering (these offer considerable potential in the future, as the dual purpose renders the products more economically viable).

The use of large PV power plants to generate electricity for the grid is not yet cost-effective.

Hybrid energy supply systems

Sometimes the reliability and cost-effectiveness of a renewable energy system can be improved by connecting two or more energy sources to the system, creating a hybrid energy system. A diesel generator in combination with PV modules will increase the ability of the system to cope with an uneven load demand profile or periods of adverse

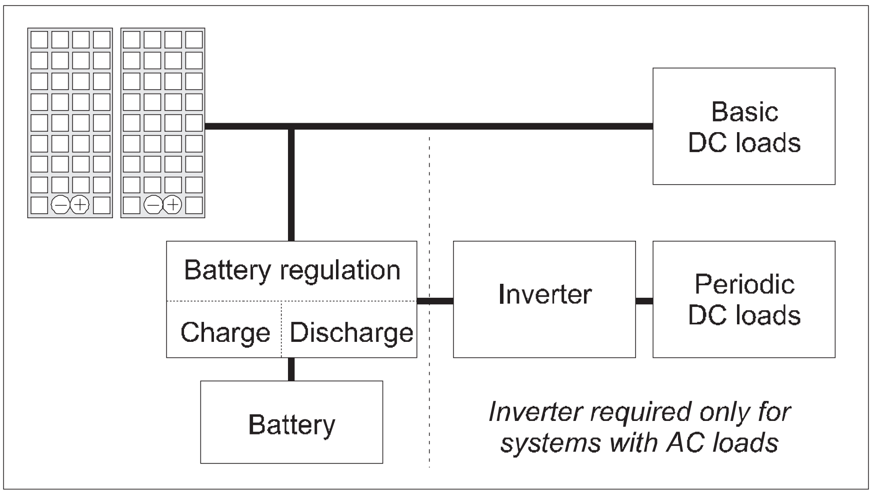

Figure 12.2.6: Principal components of a stand-alone photovoltaic system

1 The power of PV modules and system is rated in peak watts, or W_p, corresponding to the instantaneous output under standard irradiation conditions, similar to clear sun at midday.

weather conditions. Wind turbines can also complement PV systems effectively, especially in regions where periods of cloudy weather tend to be accompanied by windy weather. Hybrid systems are by nature more complex, and are thus usually justified only for larger systems (5 kWh per day and above). Design guidelines and general reference material on hybrid potential in South Africa may be found in Seeling-Hochmuth (1998). An application receiving considerable attention at present is that of remote-settlement household electrification using mini-grid reticulation systems powered by hybrid energy systems.

Costs

The modular nature of PV systems tends to make their costs per unit of energy available (kWh/day) relatively independent of system size. Most other energy systems show significant economies of scale, with diesel systems tending to be more economical where the daily power demand is greater than 2 kWh/day. PV thus tends to be most economical for smaller-scale applications. Figure 12.2.7 shows generic life-cycle cost estimates (including battery replacement, but excluding a technician's travelling time), while Table 12.2.6 indicates the range of capital costs for some common applications (including the cost of system-specific appliances).

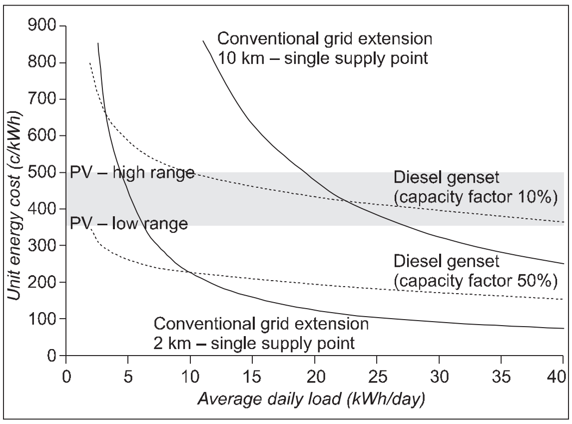

Figure 12.2.7: Comparative unit energy costs for single installations (1997 rands)

Selection of the most appropriate technology

Independent isolated applications

Capital and running costs are among the most important factors in deciding whether PV, hybrid systems, diesel, extension of the grid or other technologies are most suitable for a particular application. Costs of energy supply technologies can often be estimated reasonably accurately, and discounted life-cycle cost analyses provide an important tool to aid investment decisions. Figure 12.2.7 gives the approximate unit energy costs for PV, diesel and grid for individual applications (e.g. provision of power to a remote household or

Table 12.2.6: Typical PV applications and associated capital costs

SYSTEM DESCRIPTION	SYSTEM SIZE (PEAK WATTS PV ARRAY POWER) AND APPROXIMATE DESIGN LOAD (Wh/DAY)	CAPITAL COST (INCLUDING INSTALLATION)
Solar home system (medium): 3 DC lights, socket for B&W TV.	50 W_p 110 - 150 Wh/day	R2 500 to R4 000 depending on supplier and quantity (1997)
School system (as installed during 1996 Eskom programme): Lighting for a few classrooms, inverter to provide power for audio-visual equipment, etc.	500 W_p 1 100 - 1 500 Wh/day 900 W_p 2 000 - 2 600 Wh/day	R55 000 R61 000 (Borchers and Hofmeyr 1997)
Health Centre: Lighting, vaccine refrigeration (high availability required), examination lighting, communications; staff quarters lighting and entertainment.	450 W_p 900 - 1 200 Wh/day 600 W_p 1 200 - 1 600 Wh/day	R62 000 R76 000 (Borchers and Hofmeyr 1997)

health centre). From this figure it is clear that PV systems tend to be the most cost-effective for relatively small loads, remote from the grid. Additional factors which may increase the desirability of PV systems - even for higher load applications - include

- concerns regarding maintenance costs, reliability and security of fuel supply of diesel systems (particularly for systems where high reliability is important, such as telecommunications); and
- a requirement for low noise and/or pollution levels (particularly in environmentally sensitive areas, such as nature reserves or tourist areas).

When comparing costs of systems, it is important to consider the load devices that will be used. As will be discussed in more detail below, it is frequently worthwhile to use different appliances (and a broader range of energy carriers) than would be the case if grid electricity were available.

Settlement electrification - when to extend the grid or use off-grid technology

Where technology selection may affect an entire community, the decision is more complex. The South African electricity-supply industry is currently engaged in a large, heavily subsidised programme to extend grid access to millions of households. The rapid pace of development, and the changeable nature of planning structures, means that there is little long-term certainty regarding the future reach of the grid. Furthermore, even remote communities usually have high expectations of gaining access to the grid in the near future. As a result there is considerable reluctance on the part of communities and many institutions to invest in PV, as subsidised grid access is a far more preferable power supply option, if it can be obtained. The arrival of grid to a settlement will significantly reduce the value of any prior investment in PV or other off-grid technology. Every effort should therefore be made to determine the probability of grid electrification. Grid-planning uncertainty remains one of the most important barriers to wider-scale use of PV technology for rural electrification.

From a financial (and economic) point of view, the principal factors affecting technology choice for community electrification are listed in Table 12.2.7 However, electrification planning decisions are fraught with political, economic and social ramifications. It is thus essential that decision-makers consult appropriately, both with authorities and proposed target communities.

PV system design:

The principal components of a typical stand-alone PV system are the PV array (comprising one or more PV modules), charge controller and a lead-acid battery (see Figure 12.2.6). In some cases an inverter will be used to deliver AC power. Larger systems may incorporate a maximum power-point tracker. Both the wiring used and the specific appliances used are crucial to successful operation of a PV system. It is therefore advisable to consider the entire system from module to load appliances when conceiving and implementing a PV system application. The following aspects are important considerations for PV system design. For more detailed information on technical design elements refer to Cowan et al (1992) or Hankins (1995).

Polycrystalline, monocrystalline and amorphous (thin film) modules are available. While the latter have lower costs, they tend to suffer gradual degradation of output power over time, and should thus be used with caution in situations where long life is important.

Physical tracking of the sun can lead to gains of 25 to 35% in PV array output. However, the added complexity and reduced reliability means that this is usually not economical.

Given the high capital cost of PV systems, it is important to specify load accurately, and to avoid overdesign of capacity.

Appliance efficiency is more important for PV systems than for grid or diesel systems, due to the high marginal costs of energy.

An energy-service approach should be adopted in preference to an electricity-service approach. This requires consideration of the most appropriate energy source for each appliance/service required. For most applications involving heating PV will not be the most appropriate, as gas or other clean fuels provide a convenient and cheaper service. Refrigeration is also cheaper using gas, provided that supply availability can be assured.

The cost increases significantly if the system is required to store enough energy in the battery to deliver the full load, even under prolonged adverse weather conditions. Prioritisation of loads can be considered. Very high availability levels (requiring over-sizing) are normally only specified where essential (e.g. for vaccine refrigeration or telecommunications).

Careful matching of PV array size, storage capacity and load for local solar radiation conditions is important to reduce overall costs. Paper-based methods and a computer program (POWACOST)

Table 12.2.7: Criteria for grid versus off-grid electrification

SETTLEMENT CHARACTERISTIC VIABILITY	EFFECT ON GRID ELECTRIFICATION VIABILITY	EFFECT ON OFF-GRID ELECTRIFICATION VIABILITY
Proximity to existing or planned grid infrastructure of suitable capacity and voltage rating.	Grid extension costs are of the order of R20 000 to R55 000 per km for low and medium voltage extensions (1998).	Off-grid technology costs are little affected by distance from the grid and comparative viability thus increases with distance.
Settlement size (number of households).	Costs of grid extension per household reduce for large settlements.	Costs per household relatively independent of settlement size.
Proximity of households to each other (hh/km^2).	Costs increase for more scattered communities.	SHS costs independent of settlement layout but mini-grid systems costs are affected in the same way as grid costs.
Proximity of settlement to other similar settlements.	Clusters of settlements may be able to share bulk-electricity supply costs.	Will have little effect on off-grid electrification viability.
Expected load, especially for businesses, institutions and water supply.	Higher load can improve the viability of extending the grid.	Large loads may be most cost-effectively served by hybrid or diesel systems.
Presence or otherwise of exceptional renewable energy resources.	Scarcity of biomass may result in greater demand for electricity (for cooking and space heating).	Exceptional local wind, micro-hydro or biomass resources can offer lower cost options for mini-grid hybrid systems.
Existence in settlements of community services such as schools, health centre, community centre, police, water pumps, etc.	The economic and social benefits of electrification can be significantly enhanced.	Off-grid options can still contribute to social and economic benefits for such services, but not to quite the same extent as for grid.

are included in Cowan et al (1992).

The charge controller (or regulator) is the heart of a PV system. It is very important to match its control characteristics to those required by the particular type of battery used.

Battery technology selection is important, as incorrect matching of battery to charge controller and system load profile can lead to high battery-replacement costs. For larger systems, the use of specialist deep-cycle batteries is advised. Cowan et al (1992) provide a detailed description of battery technologies and implications for PV systems. IEC 61427-1 is a draft standard for secondary cells and batteries for PV systems.

Current flows in DC systems tend to be quite high (a 100 W load will draw 8,3 A at 12 V). Voltage drop in cables should be kept below 8%.

Both DC and AC output systems can be used. Selection of appropriate voltage will depend on the requirements of available appliances and load duty profiles. For simple systems DC is usually satisfactory. DC refrigerators are also usually more efficient. For larger systems, some appliances require AC (audio-visual equipment, workshop tools). However, the choice of DC or AC lighting on large systems is not simple. Careful analysis of the cost, efficiency and reliability implications of different options is required.

Installation of PV systems with a system voltage less than 50 V currently does not fall under the SABS 0142:1993 code of practice on the wiring of premises. However, there are indications that this will become a requirement. Cowan (1996) provides a detailed code of practice for installations.

Important applications of PV in rural settlements

Four major programmes using PV technology to deliver services in rural settlements have been initiated in South Africa. Considerable technical expertise has been built up, and although national standards are not yet available, potential users are encouraged to contact the organisations below to obtain information on standards and specifications:

- Health centre electrification: principally through the Independent Development Trust (Cape Town), with technical support primarily through the Energy & Development Group (Noordhoek).
- School electrification: Eskom (Megawatt Park and Non-Grid Electrification), with international and government support through the Department of Minerals and Energy.
- Solar Home System projects: a variety of role-players including government, Eskom and the private sector. Detailed specifications are being drafted by Eskom NRS (NRS 052, draft 1998).
- Rural telecommunications service delivery: primarily managed by Telkom.

Maintenance and user training

PV systems are generally regarded as being low-maintenance items. However, the remote location of most installations means that scheduled or unscheduled maintenance costs can be very high. Unfortunately, experience in rural community household-electrification programmes (in South Africa and internationally), and in many institutional settings such as schools and water-supply projects, indicates that provision for maintenance (financial, institutional, training, user education) is frequently insufficient. Three areas are critical:

<u>Cost of components</u>

Battery replacements, in particular, represent an ongoing cost, particularly in low-cost household applications, where average lifetimes are about 3,5 years. Lights also have a finite life (1 000 to 10 000 hours) and will need to be replaced.

<u>Availability of technical resources</u>

Although most small systems are not complex, the average user will not be able to carry out all the maintenance. Given the high cost of travelling and specialised staff, it is important to establish and support local technicians, equipped to service the systems in a given area.

<u>End-user/customer training</u>

PV systems serving loads that are under user control (households, schools, clinics, etc) need to be properly operated to obtain maximum benefit, and to reduce the probability of premature battery failure. It is thus important to provide informative user education, and to ensure that systems are equipped with indicators to inform the user regarding battery status.

Quality assurance and specifications

Although the PV industry has been active in South Africa for close on twenty years, the market size has been inadequate to develop recognised quality-assurance standards, training certification, and standard specifications for PV systems and components (other than the modules). As a result, it is sometimes difficult for the purchaser to ensure that components used are efficient and of good quality, and to ensure that design and installation are properly carried out. Changes are, however, taking place rapidly, both within the country and internationally. Standard specifications for particular system configurations and for specific balance of system components are being developed. A number of codes of practice for installation have also been developed, and may be incorporated into national standards documents. Although there has been some development of training courses, certified PV training courses are not yet well established. The following documents (or the latest drafts) are useful:

- Code of practice for installing low-voltage PV power systems (draft). Prepared for Department of Minerals & Energy, Pretoria, by the Energy and Development Research Centre (1996).
- Code of Practice: Southern African Solar Module Suppliers Association (1994).
- NRS 052:1998 (draft), Technical specification for PV systems for use in individual homes.

Finance and dissemination models

PV systems require large capital investments up front (as does grid electrification). However, the dispersed location of customers and the autonomy of installations mean that they are not normally considered part of utility business. As a result, consumers do not gain access to the low-interest - and often subsidised - long-term finance so essential to grid development, or to the project management and consumer support programmes that are crucial to normal grid-electrification project development and long-term operation.

For institutional clients such as schools, health centres and water authorities, access to finance for capital expenditure and project management can usually be resolved. Longer-term maintenance is often more difficult, as discussed above.

For households, the problems are far more intractable. Finance services geared to remote rural areas are expensive to administer, particularly for the small- to medium-scale loans required for PV systems. Incomes are often uncertain and security may be a problem. Customers require assurance that systems will work for extended periods, and there are also concerns regarding the sustained provision of maintenance services. Despite these barriers, numerous householders (numbered in the tens of thousands) have already purchased PV systems. Various strategies have been proposed or tested to enhance dissemination potential.

Hire purchase or loan schemes involve the establishment of specialised credit facilities that can be used either by individuals or, in some cases, by groups or co-operatives, to purchase PV systems. In parallel with the credit facility, NGOs, government, industry or perhaps the grid utility provide project-management services to facilitate bulk purchasing, customer education, local technician training, and efficient installation of PV systems within a community. A variation of this system is to establish a local service point near the targeted communities. This owner/franchisee/agent can then act as a liaison for loan applications, system supply and technical support. Although customers will need to make provision in the long term for ongoing maintenance costs, purchase costs are for a finite period (typically three or four years) and, once the system is paid off, monthly costs will be significantly reduced.

An alternative is to maintain more of a *utility* approach to PV electrification. In this case financial, technical and administrative infrastructure is established in a region to enable long-term leasing of PV systems by customers for a monthly fee. Essentially, the customer pays for an energy service, not a product. The approach is potentially more attractive to customers, as the risks involved are far lower, and monthly payments should be lower than for a loan-based system. Maintenance will remain the responsibility of the service provider (except for lights and possibly house wiring). Should the grid arrive at some future date, customers can simply stop their lease and return the systems to the service provider, with relatively little loss of investment.

An important technical development which makes both "service-based" and loan-based dissemination strategies simpler to administer, is the introduction of pre-payment meter technology to PV systems. This allows the user to have greater control over monthly expenditure, and provides a reasonably efficient way to eliminate bad debts. The addition of active security chips to controller, array and battery makes it possible to reduce the potential for tampering and theft.

Solar water pumping

Solar water-pumping systems use the sun's energy to produce electricity which, in turn, drives a submersible or line-shaft pump.

Solar water pumping is increasingly attractive as a replacement for hand pumps, wind pumps and diesel pumps. Between 10 000 and 20 000 systems are already in operation internationally, roughly 3 000 of which have been installed in southern Africa. Some solar pumps have been in operation for more than 10 years in South Africa.

Comparison of different pumping options

<u>Grid electricity pumps</u>

These are the cheapest option if grid electricity is available at the water source. However, extensions of MV powerlines typically cost from R40 000 to R60 000 per km and consequently non-grid options are often cheaper if grid supply is not available.

<u>Diesel water pumps</u>

This option is widely used for off-grid applications. Although the initial costs are low there are significant disadvantages to diesel systems. These include very high operating costs, high levels of maintenance and supervision, noise and dirt, and a dependence on a reliable fuel supply.

<u>Solar water pumps</u>

This option is applicable for off-grid applications and is more widely used now that farmers and authorities are becoming more familiar with the benefits of minimal maintenance, low operating costs and quiet and reliable operation. Disadvantages include the high initial costs (two or three times the costs of diesel) and the risk of theft of solar panels.

<u>Wind pumps</u>

This option has been widely used on farms and in rural areas. Approximately 300 000 wind pumps have been supplied in South Africa. Disadvantages include the relatively high levels of maintenance required and the unreliability of the wind.

Hand pumps

This option is suitable for very small water demands (<50 people). It is not suited to reticulated water-supply schemes.

Table 12.2.8 shows the comparative characteristics of these different pumping options.

When is solar pumping appropriate?

Solar pumping is appropriate in any application where grid electricity is unlikely to be available for five years (or more) and the average pumping requirement is less than 2 000 m^4/day - that is, volumetric demand (m^3/day) multiplied by the total head (m).The criterion of 2 000 m^4/day is illustrated in Figure 12.2.8.

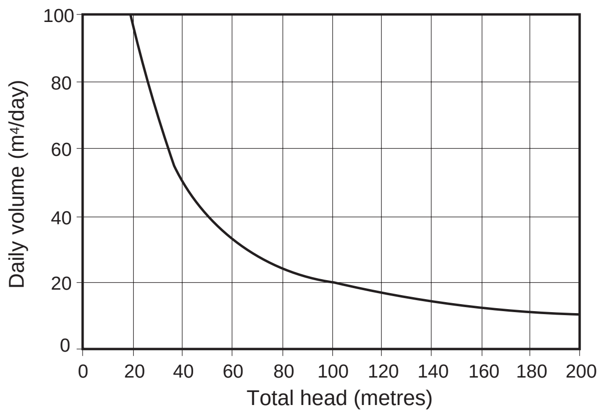

Figure 12.2.8: Volume and head relationship for the pumping criterion of 2 000 m^4 per day

Table 12.2.8: Comparative characteristics of different community water pumping options

	NO OF PEOPLE	INITIAL COST	O&M COSTS	OPERATOR INPUT	RISK OF THEFT
Grid	Any	Depends on access	Very low	None	Low
Diesel	>1500*	Low	High	High	Medium
Solar	<1500*	High	Low	Minimal	Can be high
Wind	<200	Medium	Low	Minimal	Low
Hand	<50	Low	Very low	Minimal	Low

* Assumes a 50 m total head.

Source: Borchers (1998)

Table 12.2.9: Typical initial costs and associated operating costs for different pumping systems (in 1997 rands)

NUMBER OF PEOPLE		100	500	2 000	6 000
PUMPING REQUIREMENT		125 m^4/DAY	625 m^4/DAY	2500 m^4/DAY	7500 m^4/DAY
Grid	Initial cost	7 500	8 500	11 500	16 500
	Annual cost	500	1 000	2 300	5 400
Diesel	Initial cost	36 000	39 500	46 800	63 000
	Annual cost	9 000	16 000	2 300	34 000
Solar	Initial cost	14 000	43 300	240 000	-
	Annual cost	100	200	350	-
Wind	Initial cost	15 500	-	-	-
	Annual cost	2 000	-	-	-
Hand	Initial cost	8 500	-	-	-
	Annual cost	nil	-	-	-

Source: Borchers (1998)

Costs

Typical comparative initial costs and operating costs for the different pumping options are shown in Table 12.2.9.

Principles of operation

Solar (photovoltaic) panels convert solar light energy directly into direct current (DC) electricity, which is conditioned in a controller, to an electrical output suitable for a DC or AC pump motor. Water is generally pumped into a storage reservoir and reticulated to households or community standpipes. Figure 12.2.9 Illustrates the principles of operation.

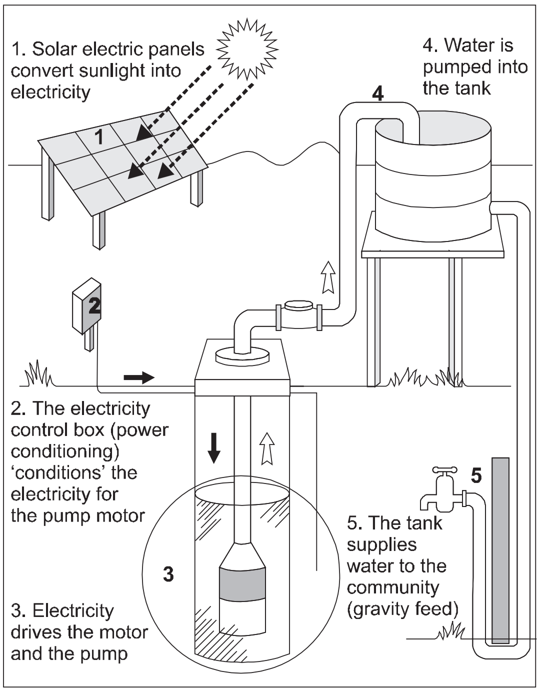

Figure 12.2.9: Operating principles of solar water-pumping systems

Description of typical solar pumping systems

There are five typical configurations of solar water-pumping systems. They are

- surface-mounted centrifugal or diaphragm suction pumps;
- floating pumps;
- submersible diaphragm or piston pumps;
- line-shaft-driven positive displacement pumps; and
- submersible multi-stage centrifugal pumps.

Three of these are illustrated in Figure 12.2.10.

Life-cycle costs of different pump types

Typical unit water-pumping costs (1998) for different solar pump options based on 15-year life-cycle costs are shown in Table 12.2.10.

Table 12.2.10: Typical pumping costs for solar pumps, based on15-year life-cycle costs

PUMP TYPE	TYPICAL PUMPING CAPACITY	UNIT COST FOR WATER
Submersible diaphragm / piston pumps	0 - 400 m4	>1,2 c/m4
Surface-mounted line shaft pumps	0 - 1 200 m4	1,5 - 5 c/m4
Submersible multi-stage pumps	0 - 4 500 m4	> 20 c/m4

Source: Borchers (1998)

Planning and installation considerations

The planning and installation considerations for solar water pumps are generally similar to other water-pumping systems, except that special care needs to be taken to minimise the risk of theft of the PV modules.

The risk of theft of the modules may be reduced through community participation and ownership, and the location of solar pumps out of sight of opportunistic thieves (i.e not near main roads).

Solar water heating

Water heating accounts for up to 40% of household energy consumption. Solar water heating (SWH) systems can provide water at temperatures of up to 85°C on sunny days, independently of other energy sources, for a wide range of uses in urban and rural areas. SWH systems generally have some form of auxiliary (or backup) heating for periods of overcast weather and for short-term peak demands which exceed the average daily output of solar-heated water.

SWH systems are an attractive alternative to more conventional water-heating systems because they have fewer negative effects on the environment and can be more cost-effective within comparatively short payback periods - as little as 3-5 years (depending on the exact application).

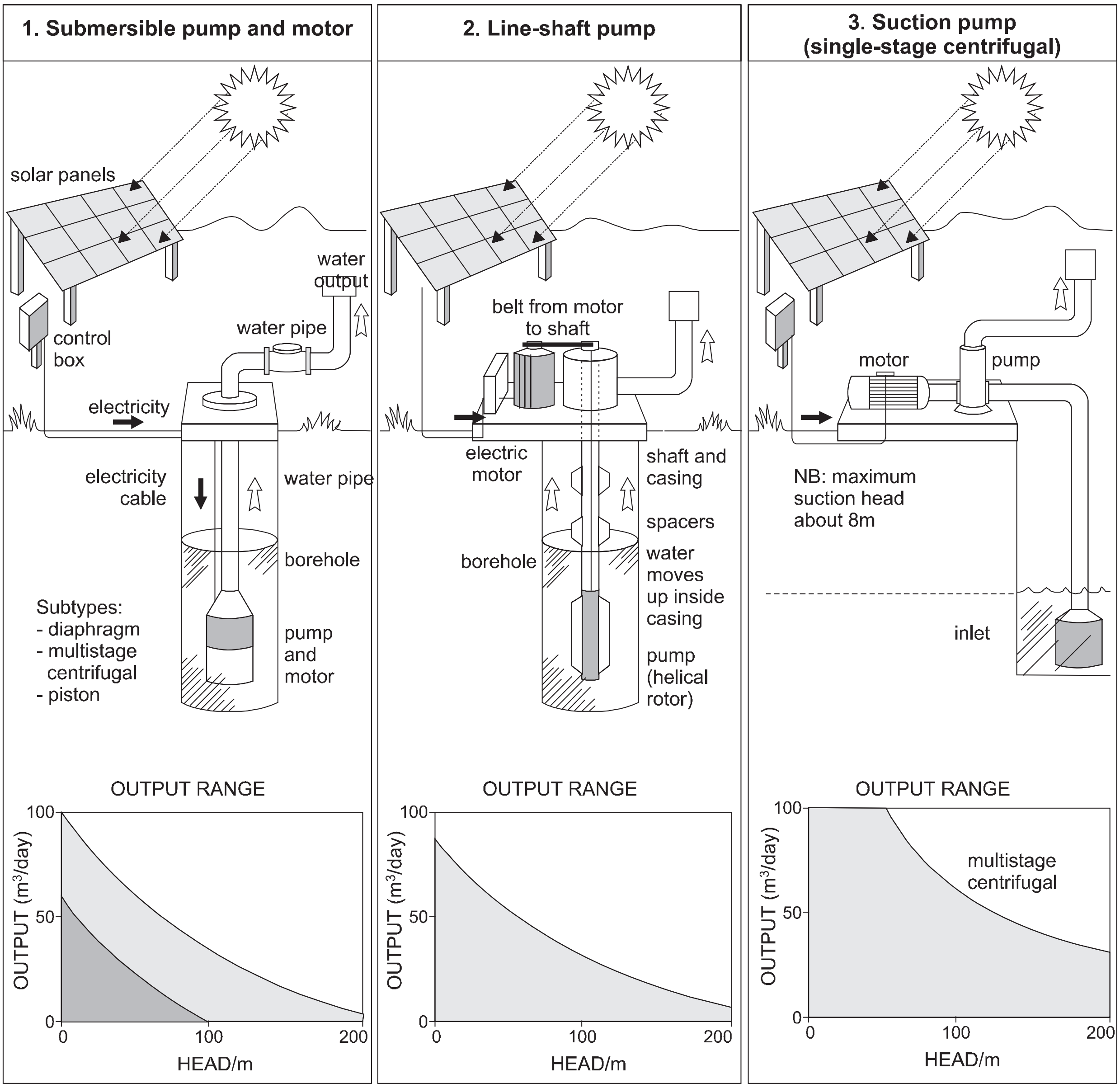

Figure 12.2.10: Three typical configurations for solar water-pumping systems

Typical applications include

- domestic water heating;
- hot water for public bath houses;
- underfloor heating;
- old age homes;
- swimming pools;
- horticulture / aquaculture;
- small businesses: laundries; hairdressers; undertakers; caterers/restaurants;
- hospitals;
- clinics; and
- school hostels.

Advantages

- the ability to provide 40-100% of hot water needs;
- the use of renewable energy (avoidance of environmental costs of electricity generation and transmission);
- the reliability of the energy source;
- the fact that it is financially cheaper (after a period of between 3-5 years); and
- the reduction of uncertainty of monthly cash flows.

Disadvantages

- the high initial cost (twice as much as a conventional electrical "geyser");
- the need for some form of backup heating for overcast weather; and
- the lack of confidence in quality assurance.

This section focuses on domestic and institutional (i.e. hostels, public bath houses, hospitals and clinics') hot-water systems.

Current use of SWH systems in SA

SWH systems have been available commercially for many years. The supply and installation of SWH systems experienced a significant peak in the mid-1970s as a result of the energy crisis and subsequent focus on alternatives to fossil-fuel-based energy systems.

SWH systems are relatively simple devices which are well suited to local manufacture. Consequently, SWH systems offer opportunities for job creation, in addition to electricity and energy savings. However, the relative ease of production requires careful consideration of quality assurance measures, to prevent poor performance and premature failure.

Current estimates suggest that, by 1994, over 70 000 domestic SWH systems had been installed in South Africa (Energy and Development Group 1997a). See Table 12.2.11.

When is SWH appropriate?

SWH systems produce hot water whenever the sun shines, regardless of whether the hot water is used. SWH systems are therefore most suited to applications which require consistent quantities of hot water on a daily basis throughout the year. Solar water heating is generally not suited to applications which require variable and unpredictable hot water demands.

In the case of households, SWH should immediately be considered if grid electricity is not available (i.e. for use in conjunction with non-grid electricity systems such as PV, wind or diesel units).

In cases where grid electricity is available, SWH systems may become cost-effective in 18 months to a few years (depending on the circumstances) for new houses and new institutional applications.

Although SWH systems operate well anywhere in South Africa, they are more appropriate in the sunny inland regions of the country.

Comparative costs

The comparative (1998) costs of SWH, electrical storage heating systems and instantaneous water heating for a typical family of four to five persons are presented in Table 12.2.12 in terms of overall life-cycle costs over 10 years.

The cumulative costs and payback characteristics are shown in Figure 12.2.11.

Table 12.2.11: Estimated solar water-heater capacity installed in South Africa

APPLICATION	INSTALLED COLLECTOR AREA (m^2)	COMMENTS
Domestic	220 000	Approx 70 000 systems
Commercial and industrial	34 000	e.g. Milnerton Girls' School
Agriculture/horticulture	2 600	
Swimming pools	227 000	Approx 8 000 systems
Public bathhouses	-	e.g. Durban Metro, Langa

Source: Borchers (1998).

Table 12.2.12: Comparative water-heating costs

	SWH SYSTEM	ELECTRICAL GEYSER	ELECTRICAL INSTANTANEOUS HEATER
Initial cost	R3 750	R2 000	R2 400
Annual operating cost	R375	R950	R750
Life-cycle costs	R5 800	R7 100	R6 600

Note: The life-cycle costs are highly sensitive to the particular application and should be established for each specific case. The key sensitivities are initial cost, real discount rate, operating lifetime, and utilisation of the solar heated water (or solar fraction).

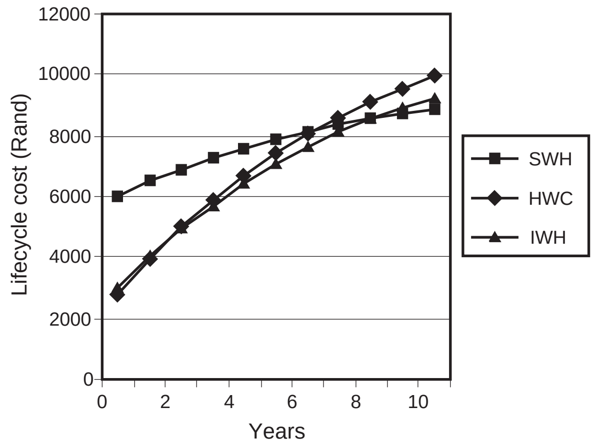

Figure 12.2.11: Cumulative costs and payback characteristics of SWH, electrical storage heaters and instantaneous water heaters

Typical considerations in planning for SWH systems

Water supply

Reticulated water supply, for a continuous/ pressurised supply, or standpipe in the yard, for a batch-heating type of supply.

Water pressure

Generally best at <200 kPa to reduce water consumption, minimise the mixing of hot and cold water in the storage tank, and reduce stress and wear and tear on the valves and connections.

Water quality implications

Generally, water which is safe for potable use is safe for use in SWH systems. Corrosion of SWH systems may be a problem with untreated (farm) water, but is generally avoided through appropriate choice of materials, such as copper and plastics.

System durability and expected lifetimes

SWH systems generally last as long as - or longer than - standard electrical storage heaters. Unless abnormal hail or freezing damages the SWH systems, the storage tank is likely to fail first.

Criteria for selecting SWH for water heating

Access to grid electricity

If the need for hot water cannot be met with grid electricity, SWH systems (with LPG or wood-fired auxiliary backup heating) are highly recommended. If grid electricity is available, the comparative costs and benefits of electrical and solar water heating need to be assessed in detail.

Hot water draw-off patterns

The water consumption patterns of the users (volume, time of draws, consistency of draw-off patterns) can influence the utilisation of solar heated water. For example, use of hot water in the afternoon and early evening is ideal (high levels of solar utilisation) while early morning consumption is less satisfactory due to overnight heat loss and the need for auxiliary backup (non-solar) heating.

Solar access

The effectiveness of SWH systems is dependent on solar access, that is, geographical location, orientation of the collectors (facing north in the range 15°E - 45°W) and angle of tilt of the collectors (ideally latitude +10°), and shading by adjacent buildings or trees. Clearly, if the collectors are not oriented correctly or if they are shaded, they will need to be oversized to compensate, thereby increasing the cost.

Freezing conditions

SWH systems are very exposed and vulnerable to freezing if they are not specifically designed for such areas (e.g. indirect heating systems). In general, all inland parts of South Africa (except the Lowveld in Mpumalanga and the Northern Province) are freezing areas. The immediate coastal zones are generally non-freezing areas.

System types

The two essential basic components of all SWH systems are the solar collector (or absorber) and the solar water-storage tank (with safety valves). Other optional components, depending on the system type, are insulated interconnecting pipework, a circulating pump, non-return valves, auxiliary backup heating systems, support structures, and instrumentation.

SWH systems may use direct heating or indirect heating systems depending on the type (see below) and the need for protection of the solar collector and interconnecting pipework against damage due to freezing. In direct-heating systems the hot water is heated directly in the solar collector, whereas in indirectly heated systems it is heated in the storage tank, using an anti-freeze primary heating fluid and a heat exchanger.

Domestic SWH systems are generally supplied in four types:

Direct batch heater

A small (10-50 litre) directly heated, portable SWH system which is filled and drawn off by hand and manually placed in the sun.

Application: Suitable for households which do not have piped water (i.e. non-pressurised) and which may not be able to afford bigger or more complex systems.

Auxiliary backup heating: None.
Typical cost: R100-R800 (1998).
Typical output: 30-120 litres/day at 55˚C for multiple draw-offs.
Suitable for freezing and non-freezing areas.

Integral SWH system

Figure 12.2.12 - a small (100-litre, typically) directly heated SWH system in which the solar water-storage tank is painted black and enclosed in a glazed and insulated casing to act as the solar collector. Integral systems are usually roof-mounted and can be pressurised. Integral SWH systems are not suitable for early morning water draw-offs, due to high overnight heat loss.

Auxiliary backup heating: Integral systems generally do not have backup heating, although electrical backup is possible.

Application: Suitable for households which have piped water but which may not be able to afford more sophisticated systems.
Typical cost: R1 200-R1 800 (1998).
Typical output: 100-150 litres/day at 55°C for multiple draw-offs.
Suitable for freezing and non-freezing areas.

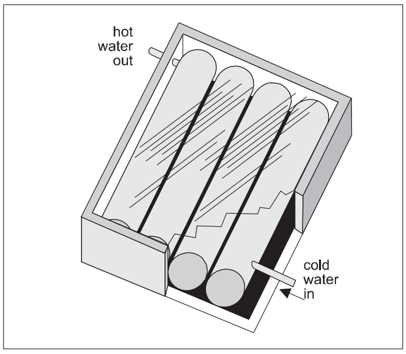

Figure 12.2.12: Integral solar water-heating system

Close-coupled SWH system

Figure 12.2.13 - a small to large (100 litre/1,6 m^2 - 300 litre/4,2 m^2) direct or indirect heating, roof-mounted SWH system in which the horizontal solar storage tank and solar collector are mounted adjacent to one another as a packaged, close-coupled unit. It may be pressurised <400 kPa and equipped with an optional single electrical backup heating element (1,5-3 kW) mounted halfway up the tank height.

Application: retrofit to existing houses or install in new houses which have flat or low-pitched roofs and will not allow a tank to be mounted in the roof space.

Typical cost: R4 000-R10 000 (1998).
Typical output: 150-400 litres/day at 55°C for multiple draw-offs.
Suitable for freezing areas (indirect heating) or non-freezing areas (direct heating).

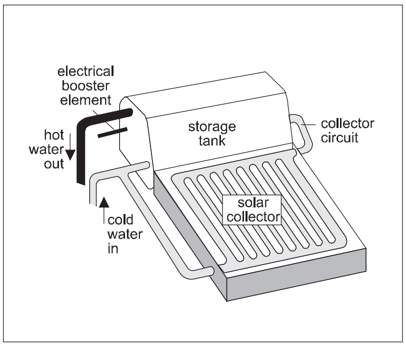

Figure 12.2.13: A close-coupled solar water-heating system

Split collector/storage system

Thermosiphon or pumped - Figure 12.2.14 - the most elegant and sophisticated of SWH system configurations, in which the solar storage tank is mounted in the roof space and the solar collector is mounted externally on the roof. These are usually larger systems (200 litre/2,8 m^2 - 300 litre/4,2 m^2) and pressurised <400 kPa. The heating fluid may circulate automatically by thermosiphon if the storage tank is physically higher than the top of the solar collector, otherwise it will require the additional complexity of a circulation pump. Electrical backup with single or dual (bottom and halfway up) elements (1,5-3 kW) forms part of the system.

Application: Aesthetic appearance most suitable for new houses with sufficient pitch in the roof to accommodate the storage tank.
Typical cost: R4 500-R12 000.
Typical output: 150-400 litres/day at 55°C for multiple draw-offs.
Suitable for freezing areas (indirect heating) or non-freezing areas (direct heating).

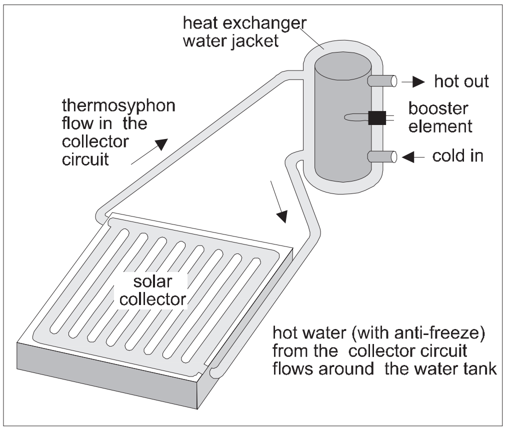

Figure 12.2.14: A split collector/storage solar water-heating system

Large commercial or institutional systems

These are all-purpose designed split collector/ storage-type systems. They are generally forced-circulation systems, although thermosiphon systems are possible if the storage tanks can be located sufficiently high above the collector bank. As in the case of domestic SWH systems, indirect heating is required in freezing areas.

Typical examples of larger systems are those at the Milnerton Girls School (Maclean 1982) and an old age home in Durban (Forbes and Dobson 1982).

Design for upgradability

In cases where the user/client is reluctant to spend the full initial cost of the SWH system, it is recommended that the (electrical) water-heating system be designed so that it can be upgraded for solar heating at a later stage.

Upgradability requires minimal extra expense (a solar-adaptable storage tank), consideration of solar access (for the future collectors), and the location of the storage tank in a position which allows short and inclining collector loop piping.

System sizing

Tables 12.2.13 and 12.2.14 provide guideline information for sizing SWH systems in typical South African conditions.

System design

System design should focus on functionality and reliability. Although thermal efficiency is important, the utilisation of SWH systems is directly dependent on reliable operation over the full design life of the system.

In the case of domestic hot water systems, there are two applicable SABS standards:

- SABS 1307:1992. Specification for domestic solar water heaters (and two associated standard test methods for physical durability testing and thermal performance testing).
- SABS 0106:1985. Draft Code of Practice for the installation of domestic solar water heaters.

Table 12.2.13: Typical hot water requirements for domestic and institutional purposes

HOT WATER SERVICE	HOT WATER SUPPLY TEMPERATURE (DEG C)	WATER TEMPERATURE REQUIRED (DEG C)	VOLUME PER DAY (LITRES/PERSON)
Domestic	55	43	50 - 75
Hospitals	65	43	120
Old age homes	65	43	100
Per shower	55	43	25 - 35
Per bath	55	43	80 - 120

Table 12.2.14: Guidelines for sizing domestic SWH systems

NUMBER OF PEOPLE PER HOUSEHOLD	LITRES OF HOT WATER REQUIRED AT 43°C	SOLAR STORAGE TANK SIZE (LITRES)	COLLECTOR AREA (m^2) DUE NORTH AT LATITUDE +10°
2 - 3	100	100	1,4 - 1,6
4 - 6	200	200	2,8 - 3,2
6 - 8	300	300	3,8 - 4,2

Other solar energy applications in South Africa

Solar water heaters, solar photovoltaic units and the passive solar design of buildings remain the most significant solar energy applications in South Africa at present.

Solar energy can also be used for process heat in industry and agriculture, but this is generally not of relevance to settlement-planning.

In the future it is possible that other means of generating electricity from solar energy will become more widespread. Brief details on solar thermal electricity generation are provided in the section "Other renewable energy sources" below.

Solar cookers

Solar stoves and solar ovens would have great relevance in South Africa if they were popularly adopted on a wide scale. They are mostly considered as a supplementary cooking option in areas of fuel scarcity, in addition to LPG, paraffin, fuelwood and crop residues. In particular, they could benefit households struggling to collect fuelwood, crop residues or dung for their cooking needs, and might help to conserve the environment.

Solar cookers generally use reflectors to concentrate radiant solar energy either directly onto a pot or into a glazed and insulated box (called a "box cooker" or "solar oven"). They are capable of cooking the typical meals of rural and low-income households. A variety of designs are available locally and internationally, including simple solar ovens, reflector-type cookers and heat pipe cookers.

Experience with the promotion of solar cookers in Southern Africa in the 1980s was disappointing. Cooking customs are deeply rooted and slow to change (except in extreme conditions - for example in refugee camps). However, a recent one-year, field testing project (Palmer Development Group 1998) undertaken on behalf of GTZ and the Department of Minerals & Energy has shown high levels of satisfaction (93%) and good utilisation (38% of all meals cooked) of a range of solar cookers.

Typical costs of the cookers in this study ranged from R180 (Sunstove) to R4 600 (SCHW1). The study found clear user preference for certain cooker types, based on utility and cost, and many of the participating households opted to buy a solar cooker.

Solar cookers are safer than fuelstoves or fires, due to the absence of a flame and greater stability.

OTHER RENEWABLE ENERGY SOURCES

This section provides a brief description of other renewable energy sources which are not widely used in South Africa at present (with the exception of wind pumps), but which may be considered in specific circumstances. The topics covered in this section are

- wind power;
- small hydropower;
- biogas and exploitation of landfill gas; and
- solar-thermal electricity generation.

Wind power

Wind power is usually employed either directly (mechanically) for water pumping, or for electricity generation.

Wind pumps

Water-pumping by windmill is a well-established technology that has long been widely used in South Africa, mainly in rural areas but also in small towns. The familiar "American farm windpump" design (with a large number of blades) is well proven, efficient for water pumping, and locally available from a number of manufacturers. More than 300 000 windpumps of this type have been installed in the country (Cowan 1992) although not all would still be in operation. These types of windpump are particularly suited to pumping relatively small quantities of water from deep boreholes, using positive-displacement pump mechanisms (e.g. for livestock watering) and can be used in areas with quite low average wind speeds (e.g. 3 ms^{-1}) but are more likely to be a cost-competitive option if wind-speeds average 4 ms^{-1} or more. Regular basic maintenance is required for trouble-free operation, and partly for this reason there has been a trend in countries such as South Africa and the United States to move towards solar PV pumps for typical ranch and game park applications (in areas without grid electricity).

The use of wind pumps for community water supplies in rural areas has so far not had a very good record in South Africa. The main problems appear to have been unreliable maintenance, lack of community involvement and ownership (Wiseman 1992), and sometimes unreliable supply/storage of water due to variations in wind energy at different times of the year, and from year to year. Any decision to use wind pumps for a community's water supply, and the design of suitable installations, should take account of

- community water needs (quantities, quality, reliability);

- the nature and reliability of the water-supply source;
- the availability of reliable wind-speed data;
- the adequacy of the local wind resource, especially in the calmest months;
- linked to this, the amount of windmill oversizing and water storage required to provide a reliable supply (or else the availability of back-up water supplies);
- water-treatment facilities;
- reliable provisions for routine maintenance and speedy repair; and
- the comparative costs and advantages/ disadvantages of alternative water-pumping options.

A good guide to wind pump sizing, design and economics is Van Meel and Smulders (1989). South African wind pump suppliers have considerable experience and can provide practical recommendations, sizings, costings, etc.

Wind-powered electricity generation

The use of wind energy for electricity generation is one of the fastest-growing developments in the world energy industry at present. Electric wind turbine generators have been used for decades for off-grid electricity (e.g. on farms) but large-scale wind farms feeding power into national grids are more recent. In areas of the world where there are favourable wind resources for electricity generation (greater than 6,5 ms^{-1} mean wind-speeds, preferably higher) and where other sources for electricity generation are expensive or inadequate, grid-connected wind farms are financially viable.

Grid-connected wind generation

Large, efficient and reliable wind turbines involve high-technology design and manufacture. The most cost-efficient machines used to be in the range of a few hundred kilowatts (rated power) but, through technical advances, now generate 1-1,5 MW. In the competitive world market for such wind turbines, there are a number of international market leaders. This leading-edge technology is considered mature and reliable, but still advancing.

Typical costs

Typical international capital costs and electricity generation costs for large-scale wind farms in 1997 were in the region of 4-5 US cents per kWh generated, in locations with very good wind resources.

The costs per kWh are highly dependent on wind speeds, because available wind power is proportional to the cube of the wind velocity. Locations for viable wind farms typically have annual wind speeds in the range 6,5-10 ms^{-1}.

Dependence on wind speeds

The total power of a wind, moving through an area of A m^2 with velocity V ms^{-1}, is given by 0,5 ρAV^3, where ρ is the density of the air in $kg.m^{-3}$ (typically 1,2 $kg.m^{-3}$ at sea-level, 1,0 $kg.m^{-3}$ at 1 500 m altitude). Not all this kinetic energy can be captured by a wind turbine. There is a theoretical maximum which can be captured - about 59% of the total power, known as the Betz maximum; the actual power captured is always less than this. Table 12.2.15 shows the dependence of maximum power on wind velocities.

Table 12.2.15: Maximum wind-power flux (Wm^{-2}) for different wind speeds

WIND SPEED (ms^{-1})	TOTAL WIND POWER (Wm^{-2})	MAXIMUM USABLE WIND POWER (BETZ MAXIMUM, Wm^{-2})
2	4	2
4	35	21
6	119	70
8	282	166
10	550	325
12	950	561
14	1 509	890

Note: For air density 1,1 $kg.m^{-3}$.

Wind turbines can be designed to obtain their peak efficiency at lower or higher wind speeds. However, this does not alter the fact that much less usable power is available from lower-speed winds.

The rated power of a wind turbine is specified for a particular wind speed (often 10 ms^{-1} or more). If a 1 MW machine is rated for a windspeed of 10 ms^{-1} it may deliver only 200 kW at a windspeed of 6 ms^{-1}, or 600 W at 4 ms^{-1}.

This strong dependence on wind speeds illustrates the vital importance of having (a) sufficiently good wind resources, and (b) reliable wind data when assessing and approving the feasibility of wind-generation schemes. A 10% error in estimating the wind resource could lead to a 30% error in estimating the costs of generating electricity.

Energy "storage" through grid connection

Available wind power at a particular site varies from hour to hour, day to day and year to year. Connection to the national grid can provide a "storage" buffer. For example, a wind farm can provide for a local municipality's needs when sufficient wind power is available, and export to the national grid when there is a surplus. When there is a deficit, power from the national grid is imported. South African wind regimes tend to be highly variable, particularly in higher-wind areas affected by cyclic weather patterns.

Financial and regulatory issues include the tariffs at which surplus wind-generated electricity will be purchased by the grid utility, the obligation on the utility to purchase surplus generation (if applicable) and the licensing conditions for the independent power producer (IPP), if applicable. None of these has yet been fully established in South Africa, but policies are on the table to pave the way for fair access to the national transmission network by IPPs.

Applicability of grid-connected wind generation in South Africa

It seems unlikely that the South African national grid, or the Southern African Power Pool, will incorporate significant wind-generation sources in the next ten years. The main reason is that electricity-generation costs from South African coal-fired power stations and from regional hydropower are less than half the best-case costs for wind generation. In the longer term, increases in the cost of coal-fired electricity generation, and pressures to rely increasingly on renewable forms of energy, may encourage more extensive use of wind energy. Pilot wind farms could be justified partly on the grounds of establishing experience for such longer-term developments.

In the short term, such pilot projects would usually require an element of subsidisation or concessionary loan finance in order to be economically sustainable. In specific situations the required subsidy element may be small - for example where:

- a municipality cannot obtain an adequate national-grid supply for less than say 25-30 cents/kWh, and/or is willing to pay a comparable amount for wind-generated electricity;

- existing transmission lines are adequate for maximum imports/exports;

- local wind-speeds average more than 7 ms^{-1} (at the hub-height of the wind turbines); and

- there is sufficient local demand for wind-generated electricity to allow for wind projects large enough to achieve economies of scale, to justify the maintenance infrastructure, etc (e.g. several MW) .

In such a situation, a detailed feasibility assessment for localised wind-farm electricity generation would be warranted. To date, no wind farms have been established in South Africa, although a promising scheme in Darling (Western Cape) is at an advanced feasibility stage.

International interests in promoting environmentally sustainable energy are favourable for obtaining "green" concessionary finance for such wind schemes. In many parts of the world where wind generation is already financially viable without any subsidy, commercial and development banks are widely engaged in wind finance.

Preliminary steps for feasibility assessments

There are two primary questions in any South African feasibility assessment for a potential grid-connected wind generation scheme:

- Are the wind resources in the identified locality sufficiently good for cost-effective wind generation?

- If so, can wind generation be cost-competitive with other electricity-supply options in this locality (taking into account any environmental subsidies for wind generation, if applicable)?

In South African conditions, it is very unlikely that grid-connected wind schemes could deliver electricity for less than the international best costs of 4-5 US cents/kWh. Therefore, if this cost is out of range for consideration, a detailed feasibility assessment is probably not warranted.

Assessing wind resources accurately is expensive and a detailed assessment for wind-farm costing and siting should preferably be undertaken by experts. Before one proceeds to this stage, however, an initial rough assessment of wind energy potential in the proposed locality can be undertaken:

- Check published wind data (e.g. Diab 1995). Is there evidence to suggest promising wind speeds in the region (e.g. an annual mean around 6 ms^{-1} or more)? Is there insufficient evidence to judge? Or is there evidence that annual mean wind speeds are generally too low?

- Check available sources of wind data for the area. How close are the nearest measuring stations? Are the measuring instruments accurate, well maintained and well exposed? Is the data sufficiently long-term to be reliable?

- Consider topography. Are there localities where wind enhancement is expected (e.g. long smooth slopes, "necks", etc)? Is the terrain too complex to make valid use of existing available data? Is turbulence a potential problem (e.g. in mountainous terrain)?

- Consider environmental acceptability (distance from dwellings, visibility impacts) and land-use.

Suitable wind data for initial cost estimates should consist at least of frequency distributions of hourly wind speeds, per month, covering a period of two years or more. These can be multiplied by the "power curves" of suitably sized wind-turbine generators, to estimate the expected electricity generation per year, leading to an initial cost estimate, and a decision whether to proceed to more detailed site-specific wind monitoring. Wind turbine manufacturers/suppliers would normally be pleased to assist.

If available wind data is not adequate for an initial assessment, preliminary site-specific monitoring can be considered, for example by installing a suitably located mast with accurate anemometers at two heights (to measure wind shear) and reliable data-logging. However, given the time implications (at least a year of data collection for preliminary assessment and more than this for detailed assessment) it is advisable to have a strategy for combining these, and expert advice would be useful. The basic techniques for initial assessment are covered well in Lysen (1982).

The broader steps in planning and implementing a wind-energy plant are covered well in the "European best practice guidelines for wind energy development", which is produced by and obtainable from The European Wind Energy Association, e-mail address 10175.1101@compuserve.com.

Off-grid wind generation

Wind power can be used for electricity supply in off-grid areas, either in the form of wind-charger/battery systems or more complex hybrid power systems (where several types of electricity-generating sources are combined - e.g. wind, diesel, solar).

In localities with average annual windspeeds above 4 m/s, even small wind turbines can generate electricity at a lower R/kWh cost than solar PV systems (Cowan et al 1992), but the problem is that the electricity supply from a wind turbine can be very variable in South African climatic conditions. In periods of high wind, much more energy is generated than in periods of low wind. Therefore, if the high-wind periods are interspersed with low-wind periods, a large amount of energy storage is needed to cover the lulls and profit from the peaks in electricity generation, in an optimal way.

In a stand-alone wind/battery system, battery storage/cycling of energy is expensive, adding at least R1-2/kWh to the electricity-generating cost. It is not cost-effective to install sufficient battery capacity to store the full amounts of electricity generated during high-wind periods, and this pushes up the average generating costs of usable energy. As a result, the net costs of off-grid electricity from wind/battery systems are expected to be higher than for PV/battery systems, according to computer modelling for the wind regimes in Cape Town, Port Elizabeth, Alexander Bay and all other major weather data stations in South Africa (Cowan et al 1992).

There is a different situation, however, if wind generators are used in a "hybrid system" configuration for off-grid power supply. By combining different sources of electricity it is possible to level out the electricity supply. To some extent, the combination of wind and solar PV generators can be complementary (especially in areas where higher winds occur during "bad" weather) but the strongest advantages occur when a controllable electricity source such as a diesel (or other engine) generator forms part of the hybrid system. This can reduce the amount of energy storage required, thus reducing costs, and provide a versatile power system for covering peaky and variable loads.

Such hybrid systems are worth considering

- for medium-load demands (e.g. 5-100 kWh/day, although there is no fixed upper limit - that will depend on the cost-competitiveness of other options, in the local circumstances);

- where the load profile is too irregular to permit the use of diesel generators at consistently high capacity factors (this pushes up the operating and maintenance costs of a simple genset power supply); and

- where skilled maintenance and diesel fuel can be provided on a regular basis.

Before including a wind generator in such a system, reasonably accurate wind-speed data should be to hand. It is likely that a partial contribution from wind generation could be technically economical if mean wind speeds are above 4-4,5 m/s. However, the extra complexity of a hybrid system (including the maintenance tasks, and the more complex system control gear required for optimal performance) should be taken into account. For more detailed information on hybrid-system design and applications, a manual and software design tool is available from the Department of Minerals and Energy in Pretoria (Seeling-Hochmuth 1998).

Applications in South Africa

Stand-alone wind-battery systems have been used in applications such as off-grid coastal holiday homes (usually less than 1 kW), and occasionally in larger applications, such as at remote schools. For larger applications, however, it is more common to combine wind generators with diesel or some other form of power back-up.

Hybrid wind-PV systems are being employed at a small number of rural clinics. Hybrid wind-diesel and wind-diesel-PV systems have been used quite extensively on off-grid commercial farms, for example in the Northern Cape.

There is considerable interest in exploring the potential of such hybrid systems to supply power for local mini-grids, in communities far from the grid. This would provide a higher level of electricity service than individual solar home systems, and may therefore be more attractive to consumers. However, the comparative costs have not yet been fully investigated. The viability is likely to depend on government policy decisions about rural electrification subsidies as well as local conditions (load density, energy resources, capacity for maintenance, etc). Hybrid systems for remote-area power supply have been particularly well developed in Australia, but are not yet common for community electricity supply in South Africa.

Small hydropower

Worldwide, hydropower is the largest application of renewable energy, producing more than 20% of the world's electricity. Within South Africa's borders the potential for large-scale hydropower is limited, due to water shortage, but the broader Southern/Central African region has great potential. It is likely that electricity from hydropower will play an increasingly important role in the Southern African Power Pool. This section, however, is concerned with small-scale applications of hydropower as an alternative to electricity from the national grid. Under suitable conditions, small hydropower plants can supply electricity at a lower cost than other off-grid electricity-supply options such as PV, wind or diesel generators.

Definitions of "small", "mini" and "micro" hydropower vary. Typical definitions (Fraenkel 1996) are

- small-scale hydropower: < 10 MW;
- mini-hydropower: < 500 kW; and
- microhydropower: < 100 kW.

Off-grid hydro plants in South African conditions are most likely to occur in the "micro" range. Eskom, for example, has focused on the development and testing of < 15 kW turbines for off-grid remote applications (Beggs 1996).

Applications in South Africa

Off-grid hydropower plants have been used for farm power and water pumping, for institutional use (e.g. electricity for boarding schools) and, to a limited extent, for local-area grids providing community electricity supply. It is estimated that several hundred micro-hydropower systems have been installed in the country. The main limiting factors are

- the availability of suitable hydrological conditions (see below);
- the ability to match the hydropower supply to local electricity demand; and
- thecomparative costs of grid electricity.

Typical configurations

Micro-hydropower configurations are usually "run of river", that is, they make use of the flow of a river and natural gradients, rather than the head of dams.

Figure 12.2.15 shows typical elements of a micro-hydropower installation. A penstock delivers water at pressure to a turbine. The penstock is often a significant cost element, requiring piping of sufficient strength and diameter to handle the water pressure, flow and shock forces. For this reason, depending on the physical characteristics of the site, it is common to construct a gradually sloping canal (gradient 1:500 to 1:1 000) from the water intake to the penstock forebay, in order to reduce the length and cost of the penstock piping. However, in some site conditions, a canal may be unnecessary or impractical.

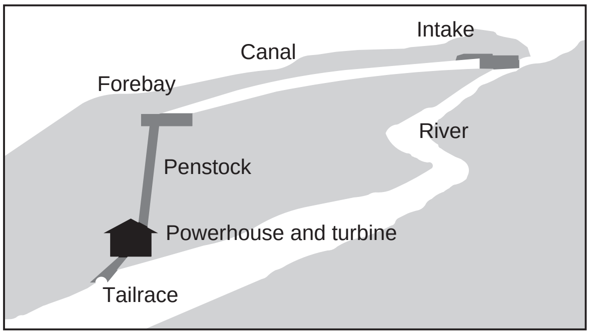

Figure 12.2.15: Schematic of typical micro-hydropower layout

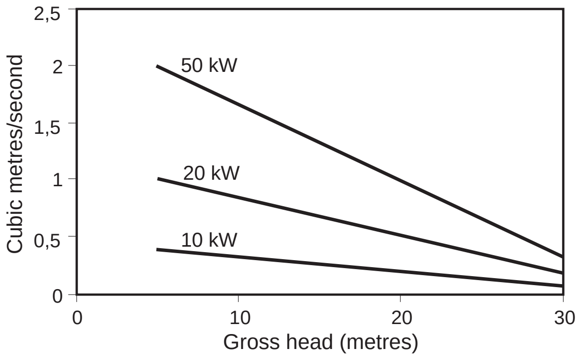

Figure 12.2.16: Approximate net electrical power as a function of flow rates and gross heads

The design of micro-hydropower installations is covered well in many texts (e.g. Harvey 1993; Inversin 1986).

Requirements for cost-effective applications

The cost-effectiveness of off-grid hydropower is highly site-specific. It depends mainly on the hydraulic power available (water flow rate and head), the civil construction required, the size and load profile of electricity demand in the locality, and (where applicable) the cost of distributing the electricity.

Hydraulic power

The gross hydraulic power P_{gross} (kW) available from a vertical head of h_{gross} (m) and volume flow rate Q (m^3s^{-1}), is given by

$$P_{gross} = 9{,}8 \times Q \times h_{gross}$$

After frictional losses, transmission losses, etc, the net electrical power P_{net} (kW) which can be delivered is usually about 40-60% of the gross hydraulic power (Harvey 1993), leading to the approximation

$$P_{net} \approx 5 \times Q \times h_{gross} \text{ (kW).}$$

Figure 12.2.16 shows approximate net electrical power for a range of flow rates and heads.

Flow rates can be measured by a variety of techniques, including

- installing a notched weir across the river - the water level above the notch indicates flow rate; and
- the “salt gulp” method - where salt water is poured into the river upstream and a conductivity meter is employed downstream to record the rate at which the salt cloud passes (Harvey 1993).

Account must be taken of variations in flow rate, requiring measurements at different times of year. Estimates of year-to-year variations can be made by looking for correlations between such measurements and longer-term records (e.g. records of water flow for gauged rivers in the same/nearby catchment area) or more detailed hydrological assessments.

Civil construction

This can include a weir for intake protection/regulation, the intake itself, a flood spillway, silt basin, canal, forebay tank with silt basin and spillway, penstock (and anchors), powerhouse and tailrace. Costs (and optimum design choices) are affected by labour costs, accessibility and cost of materials.

The proportion of total capital costs attributable to civil works will therefore vary (averaging perhaps 40% in Nepal and Sri Lanka - see Table 12.2.16) and will strongly affect viability.

Electricity demand characteristics

<u>Total demand</u>

Micro-hydropower schemes can show economies of scale. Assuming that sufficient hydraulic power is available throughout the year, the cost-effectiveness is increased for higher load demands. Conversely, if the available hydraulic power is insufficient/unreliable, more expensive back-ups may be required to meet energy needs, raising the overall cost.

<u>Load factor and capacity factor</u>

Hydropower plants have relatively high capital costs and low operating and maintenance costs. Run-of-river hydropower plants do not have significant energy storage, and are therefore most economical when the electricity generated can be fully utilised. If the combined loads are very peaky (e.g. electricity consumption mainly for domestic cooking and

Table 12.2.16: Examples of micro-hydropower capital cost proportions

	NEPAL	SRI LANKA
Penstock	12 %	21 %
Other civil works	19 %	13 %
Electro-mechanical	27 %	48 %
Engineering cost	14 %	12 %
Distribution	28 %	6 %
Total	100 %	100 %

Source: Harvey (1993)

lighting) the load factor will be low (average kW consumed ÷ maximum kW consumed) and the hydropower plant is likely to operate at a low annual capacity factor (kWh/year useful output ÷ maximum kWh/year that could be generated). Oversizing the hydropower plant will also result in low capacity factors. Low capacity factors mean higher unit energy costs, because less benefit is derived from the initial investment. Micro-hydropower viability is therefore improved if

- combined load demand is not too peaky (preferably with peaky loads smoothed out by steadier base loads);
- load demand can adjust to any seasonal variations in hydropower generation capacity; and
- plant capacity factors are high.

Electricity distribution

In cases where hydropower is used for a local grid, supplying many consumers, the electricity distribution costs are likely to be similar to conventional grid distribution. Average electricity costs per consumer, including distribution costs, will be affected by

- distance from the hydropower station and spatial density of consumers; and
- consumption levels and load factor per connection.

Large distances between consumers, with low consumption levels and low load factors, will adversely affect the economics of a hydropowered mini-grid.

In situations where a number of isolated load centres are to be connected (e.g. a number of scattered schools in a district) the use of lower-cost transmission by SWER (single wire earth return) may be indicated. Such a scheme, undertaken by Eskom Non-Grid Electrification, is reported in Dooge (1996).

Typical costs

It is difficult to give general cost guidelines for micro-hydropower electricity supply, because the costs are site-specific. Capital costs for the turbine/generator can be in the region of R3 000/kW (1998 estimate). An indication of the breakdown of the total capital costs (based on figures from Nepal and Sri Lanka) is given in Table 12.2.16.

In 1992, typical South African micro-hydropower unit electricity costs, levelised over a 20-year system life, were estimated as given in Table 12.2.17.

These figures should be inflated by 60-100% to give comparable 1998 estimates. They still show that micro-hydropower can be the cheapest off-grid electricity source, in suitable conditions.

Table 12.2.17: Typical supply costs of micro-hydroelectric power (1992 estimates, excluding reticulation)

HEAD/SLOPE	DAILY DEMAND	CAPACITY FACTORS	
		0,25	0,4
Head 10 m Slope 1:10	5 kWh 25 kWh	74 c/kWh 30 c/kWh	68 c/kWh 23 c/kWh
Head 10 m Slope 1:50	5 kWh 25 kWh	95 c/kWh 39 c/kWh	84 c/kWh 32 c/kWh

Source: Cowan et al (1992)

BIOGAS AND LANDFILLS

Biogas and landfill gas are both potential sources of energy for heating and lighting applications in South Africa. In both cases, methane gas (also known as marsh gas) is produced from a process of digestion of organic waste material. This methane gas can be collected, stored and reticulated for use in households and small businesses. The gas may be used directly in suitably adjusted appliances or indirectly in gensets for electricity generation.

Biogas is highly dependent on a reliable and suitable supply of organic waste (animal manure and plant material) and water. Landfill gas is available only from landfill sites.

Consequently, neither option is generally applicable but there are a number of successful applications which could be replicated. Many municipal sewage works produce biogas as a by-product of the treatment process. This biogas is usually used on site, and not supplied to the surrounding communities. Interestingly, communal household biogas digesters have been widely used in China to handle the digestion of human waste (primarily) and to produce biogas for heating and cooking.

In the case of landfill gas, there are at least two South African applications of note. These include the Grahamstown landfill project, which supplies an adjacent brick kiln, and one of the Johannesburg landfills which provides methane to the chemical industry.

Biogas production

Biogas is produced in a purpose-built biogas digester. The digester comprises an airtight container into which raw waste can be poured, together with water, as a slurry and from which methane gas can be collected under some pressure. There are many configurations of digester, including the underground “Chinese”, or fixed-dome digester, and the “Indian” or floating-dome, digester.

The methane is produced by anaerobic digestion of organic matter. The production of methane is highly sensitive to temperature and increases dramatically at higher than ambient air temperatures. The added complexity of auxiliary heating of the digester cannot always be justified by the extra production obtained.

Landfill gas production

Landfill gas is automatically produced in landfills as a result of decomposition of the organic content of municipal refuse collections. This gas is a potential explosion hazard if it is not managed properly and can be utilised productively by creating sealed landfill compartments and extraction wells (similar to boreholes)

Solar-thermal electricity generation

Photovoltaic panels provide a simple, low-maintenance option for generation of electricity from the sun. However, as discussed in the section on solar photovoltaic electricity supply, the per-unit energy costs are relatively high, and there is little size-related reduction in cost as plant size increases. A potentially lower-cost alternative for both small- and large-scale grid-connected plants is to use concentrators to focus sunlight to heat a working fluid, which is then used to drive a conventional thermodynamic power cycle. Thermal-energy storage and/or supplementation with gas-fired burners can be used to deliver power over extended periods. Given the high levels of solar radiation available in parts of South Africa, there has been considerable interest in the possible development of such large-scale grid-connected plants. At present, however, the generation costs would be higher than for South African coal-fired power stations.

Four main technology options are potentially suitable for grid connection. Parabolic dish systems also have the potential to be used in smaller-scale applications (5 kW or larger), either as part of a grid-connected distributed generation system, or for mini-grid or stand-alone applications.

Parabolic trough-based systems

These systems focus sunlight energy onto heater pipes contained within evacuated tubes located along the focal axes of parabolic troughs. Insulated piping connects a field of troughs to a steam generator (with natural gas backup), used to drive a conventional Rankine Cycle turbine-generator. This technology is well established, with a number of plants having been installed. A series of eight plants with a total output of 354 MW has been operating since the late 1980s in California. Demonstrated peak efficiencies of 20% have been achieved.

Power-tower systems

These systems use an array of heliostats (large, individually tracking mirrors) to focus sunlight onto a receiver located at the top of a tower. Here the energy can be used either to generate steam to drive a Rankine Cycle, or to heat air to drive a

Brayton Cycle machine. Several power-tower plants have been established, with the recently commissioned (1996) "Solar Two" project demonstrating integration of molten-salt thermal storage technology.

Dish-Stirling or Dish-Brayton cycle machines

These systems are generally much smaller (of the order of 5-50 kW) per machine. A Stirling or Brayton (gas turbine) machine is mounted at the focal point of a parabolic dish. A large power plant would require a field of such engines to be installed. They are, however, potentially well suited to distributed generation or even stand-alone applications, such as for a mini-grid or for specific higher load applications in rural areas. Dish-Stirling systems currently hold the record for system solar-to-electricity conversion efficiency at 29,4% and can generate AC power directly. The Stirling-engine generator unit also has the potential to be powered by gas or biomass. This allows greater productivity of the capital invested and the potential for night-time or weather-independent power, without relying on expensive battery storage.

Solar chimney

This somewhat unusual approach involves construction of an enormous glass-covered expanse, with the exterior perimeter open and surrounding a tall chimney (perhaps 750 to 1500 m high in current conceptual designs being considered in South Africa). Solar radiation would heat the air trapped under the glass, lowering its density. As a result it would then rise up in the chimney, generating wind speeds in the chimney of the order of 15 ms^{-1}. Wind turbine technology can then be used to generate power. Unlike the technologies described above, solar chimney plants are expected to have low efficiencies (of the order of 1%).

None of the above technologies is currently at a stage where commercial utilisation in South Africa could be justified on financial or economic grounds. However, given the excellent solar resource in parts of the country, the potential reductions in cost that are expected for some of these technologies, and increasing environmental concerns regarding fossil-fuel utilisation, the future prospects for solar thermal power generation may be good. Particular niche markets include regions remote from the grid, or sites where transmission costs result in added cost for conventionally generated power. EPRI & DOE (1997) provide an excellent review of the technical and economic status of solar thermal technologies, from which much of the above material has been drawn.

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