Federal Republic of Nigeria

Federal Ministry of Works

Highway Manual Part 1: Design

Volume IV:

Drainage Design

2013

DRAINAGE DESIGN

FOREWORD

The vision statement of the Federal Ministry of Works is to elevate Nigerian roads to a standard where they become National economic and socio-political assets, contributing to the Nation's rapid growth and development, and to make Federal roads functional, safe, pleasurable, and an avenue for redeeming Nigerians' trust and confidence in Government. This vision statement is in tune with the Transformation Agenda of the President of the Federal Republic of Nigeria, His Excellency, Dr Goodluck Ebele Jonathan, GCFR. Based on the foregoing, our mission is to use the intellectual, management and material resources available to the Ministry to make Nigerian roads functional all the time. The principal goal of the Ministry is to drive the transformation agenda by improving road transport infrastructure for the overall socio-economic derivable benefits and development of our great country, Nigeria.

In exercising this mission and in discharging its responsibilities, the Ministry identified the need for updated and locally relevant standards for the planning, design, construction, maintenance and operation of our roads, in a sustainable manner. One of the main reference documents for this purpose is the Highway Manual, which previously included Part 1: Design and Part 2: Maintenance. Both current parts of the Highway Manual were first published in 1973 and 1980 respectively and have been subjected to partial updating at various times since then. The passage of time, development in technology, and a need to capture locally relevant experience and information, in the context of global best practices, means that a comprehensive update is now warranted.

The purpose of the Highway Manual is to establish the policy of the Government of the Federal Republic of Nigeria with regard to the development and operation of roads, at the Federal, State and Local Government levels, respectively. In line with this objective, the Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deem acceptable; to direct practitioners to other reference documents of established practice where the scope of the Manual is exceeded; to provide a nationally recognized standard reference document; and to provide a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

The major benefits to be gained in applying the content of the Highway Manual include harmonization of professional practice and ensuring uniform application of appropriate levels of safety, health, economy and sustainability, with due consideration to the objective conditions and needs of our country.

The Manual has been expanded to include an overarching Code of Procedure and a series of Volumes within each Part that cover the various aspects of development and operation of highways. By their very nature, the Manual will require periodic updating from time to time, arising from the dynamic nature of technological development and changes in the field of Highway Engineering.

The Ministry therefore welcomes comments and suggestions from concerned bodies, groups or individuals, on all aspects of the document during the course of its implementation and use. All feed back received will be carefully reviewed by professional experts with a view to possible incorporation of amendments in future editions.

Arc. Mike Oziegbe Onolememen, FNIA, FNIM.

Honourable Minister

Federal Ministry of Works,

Abuja, Nigeria

May, 2013

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Nigeria Police Force (Traffic Division)
Nigeria Hydrological Services Agency
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This update of the Highway Manual was compiled by the Road Sector Development Team of the Federal Ministry of Works with the assistance of the consultants Royal HaskoningDHV.

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1: GENERAL INFORMATION

1.1 Description of the Manual

The Highway Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deems acceptable for the planning, design, construction, maintenance, operation and management of roads. The Manual directs practitioners to other reference documents of established practice where the scope of the Manual is exceeded; provides a nationally recognized standard reference document; and provides a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

1.2 Arrangement of the Manual

The Highway Manual comprises a Code of Procedure and two Parts, each of which has been divided up into separate volumes, in the manner shown in **Error! Reference source not found.**.

1.3 Overview of Volume IV: Drainage Design

1.3.1 General

Volume IV of the Highway Manual Part 1: Design deals with the design of drainage elements of highways.

1.3.2 Purpose

The purpose of this volume dealing with drainage design is to give guidance and recommendations to the engineers responsible for the design of Federal Highways and other roads in Nigeria.

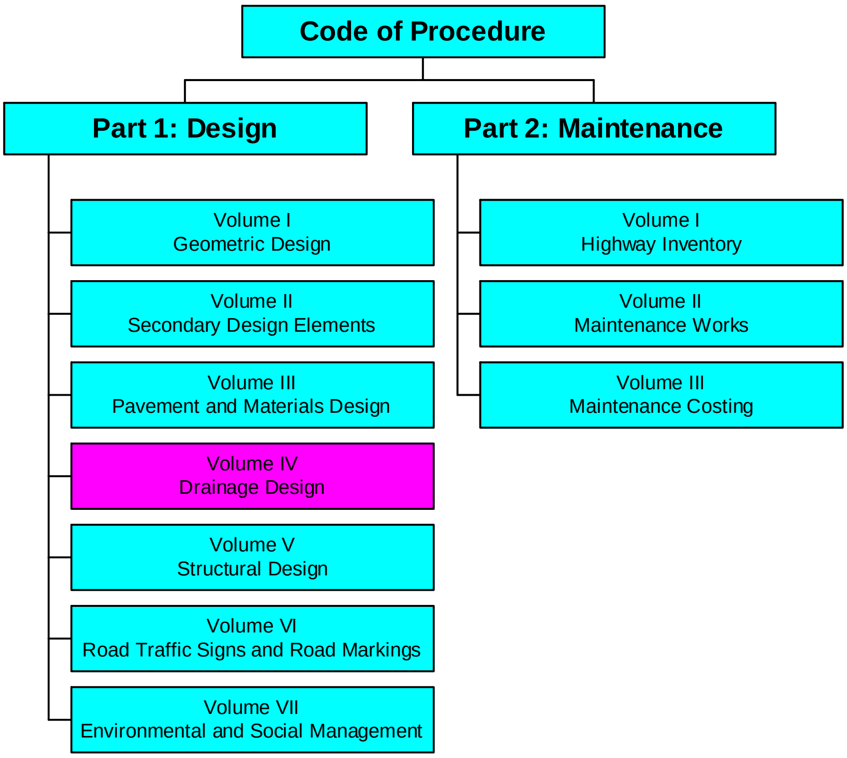

Figure 1.1: Organisation of the Highway Manuals showing context of Volume IV

1.3.3 Scope of this volume

This volume has been developed by the Federal Ministry of Transportation (Works) with the intent to reflect and establish uniform policies and procedures for planning and designing highways. The procedures presented in this volume are applicable to all classes of Federal roads, and the principles may be applied to equivalent classes of road in any situation in the country.

The contents of the volume are partly guidelines and recommendations and partly standards which as a general rule should be adhered to. The information, guidance and references contained in this volume are not intended as a substitute for sound engineering judgment. It should be recognized that situations may be encountered during the design of highways that are beyond the scope of this volume. Relevant supplementary references are listed at the conclusion of each chapter, as well as in Appendix 5. These should be used where additional information is required. In some instances special conditions may require the use of other references and/or standards and the use of these standards can only be sanctioned by the Director of Planning and Design of the Federal Ministry of Transportation (Works).

1.3.4 Terminology

Figure 1.2 and Figure 1.3 present commonly used terminology applicable to the various elements of a typical road.

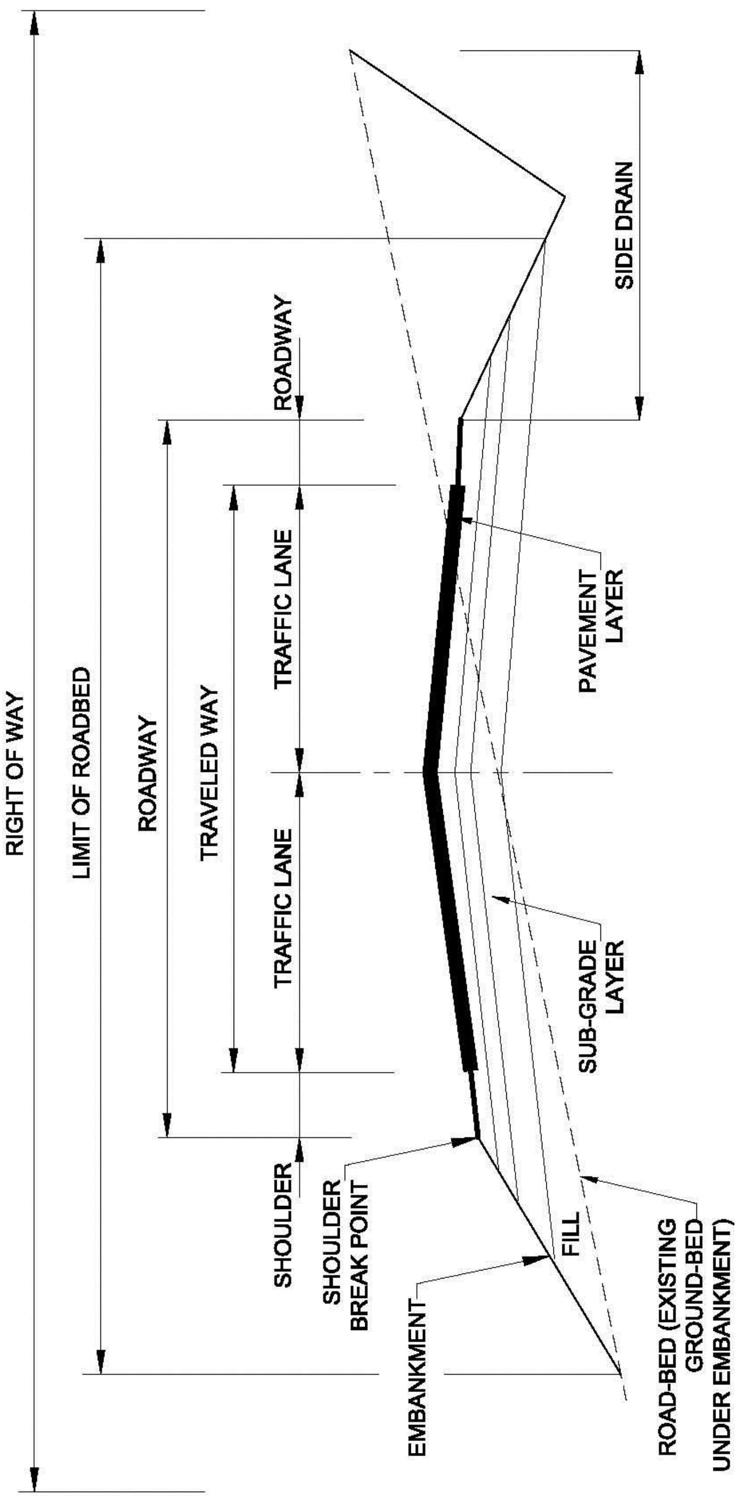

Figure 1.2: Road Cross-Section Elements

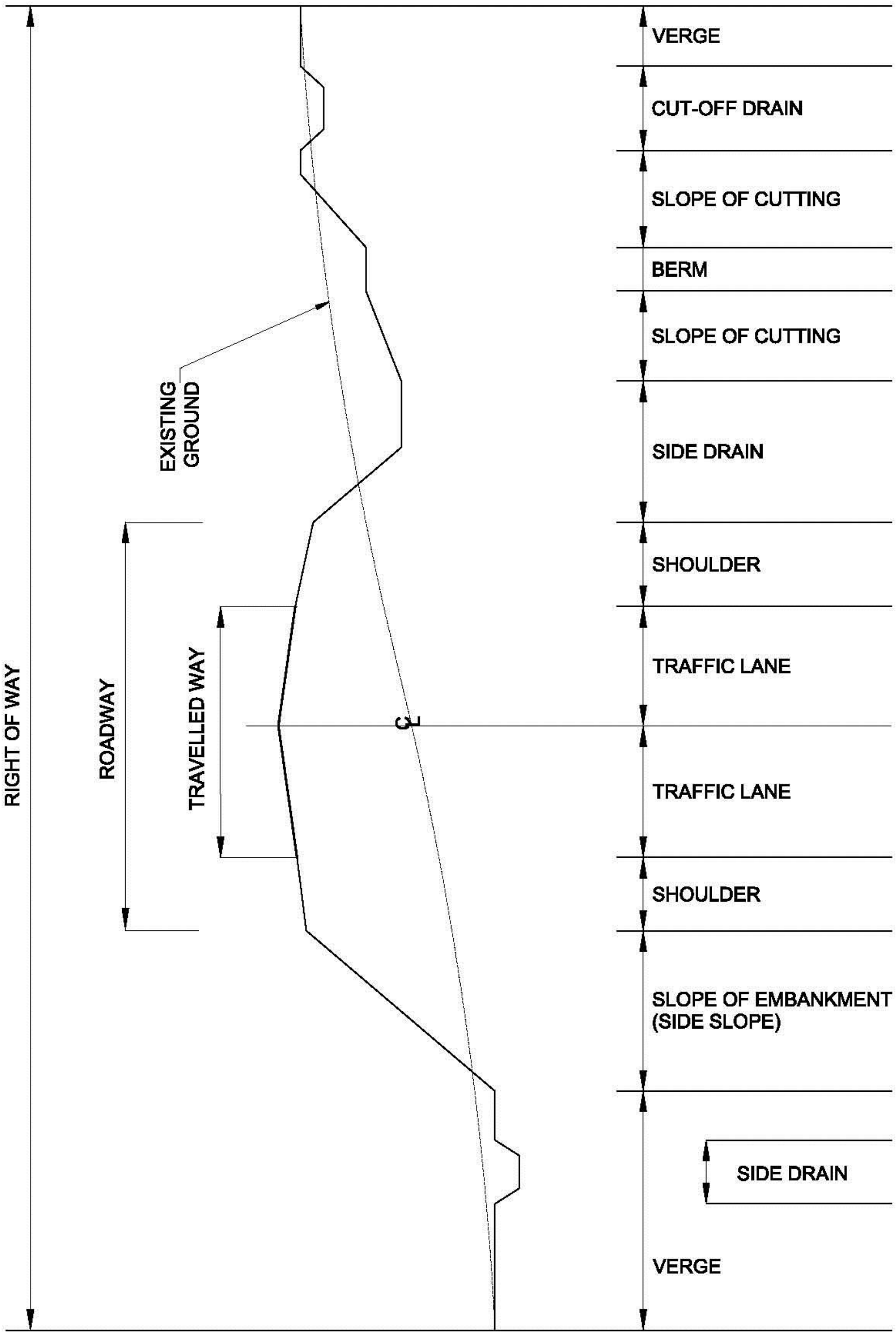

Figure 1.3: Cross-Section Terminology

2: POLICY ON DRAINAGE DESIGN

2.1 Introduction

Stormwater is a component of the total water resources of an area and should not be casually discarded but rather, where feasible, should be used to replenish that resource. In many instances, stormwater problems signal either misuse of a resource or unwise land occupancy.

There should, therefore, be an increasing awareness for reiteration of approaches to basin-wide management, keeping in mind the likely ultimate state of development when considering the peak flows. Although traditional drainage concepts of the past allow upstream development to increase runoff, with the consequence that downstream development has to accommodate, sometimes at significant additional cost, the upstream excess runoff, the more recent trend is to require that, for design rainfall frequencies (up to 10 years), the peak runoff should not be significantly different after development of an area than it would be if such development had not taken place. This needs to be taken into consideration in road drainage design, both in determining design flows from upstream catchments, but also in discharge patterns through the structures, resulting in changed downstream flows.

2.2 Design Objectives

The objective of highway drainage design is to provide for the safe passage of vehicles during the design storm event, while mitigating any adverse stormwaterstormwater runoff impacts on properties outside the right of way as a result of the road construction. The drainage system is designed to collect stormwater runoff from the roadway surface and right-of-way, convey it along and through the right-of-way, and discharge it to an adequate receiving body without causing negative on- or off-site impacts.

Stormwater collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Rapid removal of stormwater from the pavement minimizes the conditions, which can result in the hazards of hydroplaning. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity.

The objective of stormwater conveyance systems (storm drain piping, ditches and channels, etc.) is to provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without surcharging inlets or otherwise causing surface flooding. Erosion potential must also be considered in the design of open channels or drains used for stormwater conveyance.

The design of appropriate discharge facilities for stormwater collection and conveyance systems includes consideration of stormwater quantity and quality. This is particularly relevant in urban areas where there may be insufficient capacity in the downstream drainage system to receive the expected design flow (quantity) and in sensitive natural environments where the discharge of road-generated contaminants may cause unacceptable ecological damage. To address these particular issues, storm drainage systems will usually require detention or retention basins, and/or other best management practices for the control of discharge quantity and quality.

2.3 Basic Policy on Drainage Design

Improvement of the effectiveness of natural systems rather than replacing, downgrading or ignoring them is an objective of current engineering design. In this regards, the basic principles followed in the design of road drainage includes the following.

In areas where soil and physical conditions permit, the road shall be drained directly into the road reserve.

Where natural watercourse and drainage channels exist, the road reserve shall be drained directly into them.

Where conditions necessitate drainage beyond the reserve, additional land shall be acquired for the necessary drainage channels, but this should be the exception rather than the rule.

In urban areas where drainage channels have been provided by other authorities, the Director Highway Planning and Design and Director Highway Construction and

Rehabilitation shall participate in the overall project subject to a formal agreement to the extent of the amount of discharge from the highway.

The Directors of Highways Planning and Design, and Construction and Rehabilitation may participate in flood control projects (including detention) in the interest of the highway. The amount of participation shall be based on the legal premise that aid must be restricted to the amount of benefit accruing to the Federal Government by reason of the improvement. Such projects shall be covered by a formal agreement.

Two principal systems for handling surface water runoff are recognized. The one on which engineering planning, design and operations have been almost wholly concentrated (the "Minor System", equally called the "Convenience System") and the larger storm drainage system (the Major System) which includes all the natural and man-made drainage facilities in an entire watershed, including the major river crossings on highways.

The "Minor System" is the scheme of kerbs, gutters, inlets, pipes or other conveyances, channels, drains and appurtenant facilities all designed to minimize nuisance, inconvenience and hazard to persons and property from storm runoffs which occur at relatively frequent intervals. Currently more detailed attention is also being given to the planning and design of the supplementary aspects of the overall "major system" which carry the excess flow over and above the hydraulic capacity of the various components of the minor system.

The increased use of storage to balance out handling or treatment of peak flows; use of land treatment systems for handling and "disposal" of stormwater; and perhaps most important, a recognition of detention at various points in a system, are potential design solutions rather than problems in many situations.

Another basic reality is that every site or situation presents a unique array of physical resources, land use conditions and environmental values. Variations of such factors generally require variation in planning of approaches and standards for optimal achievement of stormwater management objectives.

A distinction is drawn between rural and urban drainage. Rural drainage focuses largely on the swift removal of stormwater from the travelled way onto the verge and on its movement to a point where it can be taken from the upstream to the downstream side of the road. In an urban environment, the road reserve serves as the principal conduit of stormwater from surrounding properties and its conveyance to a point where it can be discharged into natural watercourses.

Since many communities and urban areas use less than a 10 years frequency value for their storm drainage facilities, coordination of the highway drainage with that of the local urban area is a primary factor requiring very careful consideration. Location studies of a highway through a built up area require close attention to how the proposed highway's drainage requirements can be satisfactorily coordinated with those of the community. Necessarily, both horizontal and vertical location of the proposed highway improvements are of great significance since most major city streets are likely to have existing storm sewers and underground utilities.

In both rural and urban environments, stormwater drainage is aimed both at the safety of the road user and the integrity of the design layers of the road. The safety of the road user dictates that a maximum depth of drain be specified and the recommended maximum depth is 500 mm. The volume of water to be conducted by a drain thus indicates the required width of the drain rather than its depth, since the need to protect the design layers from saturation by both stormwater and subsurface water remains unchanged.

In addition to the control of stormwater quantity through the provision of stormwater detention facilities, current international best practice also focuses on the provision of best management practices (BMPs) for the treatment of stormwater quality. Such practices can often be provided at lower cost than conventional hard engineering solutions and should therefore be considered as part of the overall design philosophy. In sensitive natural environments, such practices may be required as part of a series of mitigating measures to comply with the environmental approval process.

2.3.1 Rural Drainage Design

a. Elements of rural drainage

Rural drainage is normally by means of unpaved open drains, which may either silt up or scour, depending on the typical flow velocity. Both siltation and scouring of a drain increase the hazard to the road user. Scour would lead to the creation of a deep channel that would be impossible to traverse with any degree of safety. It may also cause erosion of the shoulder and ultimately threaten the integrity of the travelled way itself. Siltation may block the drain, so that water that should have been removed would be discharged onto the road surface.

The effectiveness of the drain depends on the design flow velocity, which is a function of longitudinal slope, as well as other variables. There is a range of slopes over which the flow velocity of water on in-situ materials will be so low that siltation occurs, and another range where the flow velocity will be high enough to cause scour. On slopes between these two ranges neither siltation nor scouring will occur and unpaved drains will be effective.

Paving solves some of the problems caused by both siltation and scouring. Paving generally has a lower coefficient of roughness than in-situ materials, so that water speeds are higher in a paved drain than in an unpaved drain with the same slope. Furthermore, it is possible to force higher speeds in the paved drain by selection of the channel cross- section. The flow velocity below which siltation is likely to occur is 0.6 m/s. Flow velocities above which scour is likely to occur are presented in Table 5.2 and Table 5.4 in Chapter 5. Conventional open-channel hydraulics will, in conjunction with this Table, indicate when either siltation or scouring is likely and hence whether it is necessary to pave a drain or not.

As a rough guide to longitudinal slopes, it is suggested that unpaved drains should not be steeper than 2%, or flatter than 0.5%. Paved drains should not be flatter than 0.3%. Practical experience indicates that it is difficult to construct a paved drain accurately to the tolerances demanded by a slope flatter than 0.3%, so that local imperfections may cause siltation of an otherwise adequate drain. Where the longitudinal slope is so flat that self-cleansing water speeds are not achieved, even with paving, it will be necessary to consider a piped drainage system.

As an alternative to lining a drain in material subject to scour, it is possible to reduce flow velocity by constructing weirs across an unpaved drain. The drain will then in effect become a series of stilling basins at consecutively lower levels. While this could be an economical solution in terms of construction cost, it has the disadvantage that an area of deep localised erosion, immediately followed by a stone-pitched or concrete wall, would confront an errant vehicle. If this alternative is to be considered at all, it should be restricted to roads with very low traffic volumes and the weirs should be spaced as far apart as possible. Drains constructed through in-situ materials generally have flat inverts so that, for a given flow, the flow velocity will be reduced. The flat inverts reduce the possibility of scour and are easy to clear if siltation occurs. Paved drains, not being susceptible to scour, have a V-profile. Self-cleansing velocities are thus achieved at relatively small flows and the need for maintenance is reduced.

The sides of the drain should not be so steep as to be dangerous to the road user; a maximum slope of 1:4 is recommended. Ideally, both sides of the drain should be designed to this slope or flatter. Where space for the provision of the drain is restricted, the slope closest to the road should remain at 1:4 and the outer slope steepened. This has the effect of positioning the drain as far as possible from the path of vehicles. One example of this is a side drain in a cut, where the outer slope of the side drain forms an extension of the cut face. These slopes, in combination with the flat invert, give the trapezoidal profile for an unpaved drain.

It is recommended that the bottom of a lined V profile and the junctions between the sides and bottom of an unlined trapezoidal profile be slightly rounded. The rounding will ease the path of an errant vehicle across the drain, and reduce the likelihood of a vehicle digging its front bumper into the far side of the drain and somersaulting.

(i) Side drains

Side drains are located beyond the shoulder breakpoint and parallel to the centre line of the road. While usually employed in cuts, they may also be used to run water along the toe of a fill to a point where the water can conveniently be diverted, either away from the road prism or through it, by means of a culvert. When used in conjunction with fills, side drains should be located as close to the edge of the reserve boundary as is practicable to ensure that erosion of the toe of the fill does not occur. Side drains are intended as collectors of water and the area that they drain usually includes a cut face and the road surface.

(ii) Edge drains

Edge drains are intended to divert water from fill slopes that may otherwise erode either because of the erodibility of the material or because they are subjected to concentrations of water and high flow velocities. Guardrail posts tend to serve as points of concentration of water, so that, as a general rule, edge drains are warranted when the fill material is erodible or when guardrails are to be installed.

Edge drains should preferably be raised rather than depressed in profile. A depressed drain located almost under a guardrail would heighten the possibility that a vehicle wheel might snag under the guardrail. Edge drains are constructed of either concrete or premixed asphalt. Premix berms normally have a height of 75 to 80 mm, and are trapezoidal in profile with a base width of 250 mm and a top width of 100 mm. Concrete edge drains are normal barrier kerbs and channels. These require a properly compacted backing for stability and are, therefore, not as easy to construct as premix berms.

(iii) Catch water drains

The catch water drain, a berm located at the top of a cut, is to the cut face what the edge drain is to a fill. It is intended to deflect overland flow from the area outside the road reserve away from the cut face. Even if the cut is through material which is not likely to scour, the catch water drain serve to reduce the volume of water that would otherwise have to be removed by the side drain located at the bottom of the cut face. Catch water drains are seldom, if ever, lined. They are constructed with the undisturbed topsoil of the area as their inverts and can readily be grassed as a protection against scour. Transverse weirs can also be constructed to reduce flow velocities, since the restrictions previously mentioned in relation to weirs do not apply to catch water drains. The cut face and the profile of the drain reduce the probability of a vehicle entering the drain but, should this happen, the speed of the vehicle will probably be low.

(iv) Median drains

Median drains do not only drain the median but also, in the case of a horizontal curve, prevent water from the higher carriageway flowing in a sheet across the lower carriageway. The space available for the provision of median drains makes it possible to recommend that the transverse slopes should be in the range of 1:4 to 1:10. If the narrowest median recommended is used, a transverse slope flatter than 1:10 may make it difficult to protect the design layers of the road. Unlike side drains, median drains, whether lined or not, are generally constructed with a shallow V-profile with the bottom gently rounded.

(v) Chutes

Chutes are intended to convey a concentration of water down a slope that, without such protection, would be subject to scour. They may vary in size from large structures to half-round precast concrete products, but they are all open channels. Flow velocities are high, so that stilling basins are required if down-stream erosion is to be avoided. An example of the application of chutes is the discharge of water down a fill slope from an edge drain. The entrances to chutes require attention to ensure that water is deflected from the edge drain into the chute, particularly where the road is on a steep grade. It is important that chutes be adequately spaced to remove excess water from the shoulders of the road. Furthermore, the dimensions of the chutes and stilling basins should be such that these drainage elements do not represent an excessive risk to errant vehicles. Generally, they should be as shallow as is compatible with their function and depths in excess of 150 mm should be viewed with caution.

Because of the suggested shallow depth of chutes, particular attention should be paid to their design and construction to ensure that the highly energised stream is not deflected out of the chute. This is a serious erosion hazard that can be obviated by replacing the chute with a pipe.

(vi) Mitre banks

As their name implies, these banks are constructed at an angle to the centre line of the road. They are intended to remove water from a drain next to the toe of a fill and to discharge it beyond the road reserve boundary. Several mitre banks can be constructed along the length of a drain, as the concentration of water in the drain should ideally be dispersed and its speed correspondingly reduced before discharge. Speed can be reduced not only by reducing the volume, and hence the depth, of flow but also by positioning the mitre bank so that its toe is virtually parallel to the natural contours. The upstream face of a mitre bank is usually protected by stone pitching, since the volume and speed of flow of water that it deflects may cause scour and ultimately lead to breaching of the mitre bank.

b. Rural underground systems

The geometric designer is not directly concerned with the underground system, except for its inlets. These should be hydraulically efficient and correctly positioned to ensure that water does not back up onto the road surface or saturate the design layers. To restrict the hazard to the road user, inlets that are flush with the surface drain invert are preferable to raised structures. Underground reticulation is costly both to provide and to maintain. The designer should therefore, without violating the principles discussed above, attempt to reduce the use of underground drainage as far as possible by the discerning use of surface drainage.

2.3.2 Urban Drainage Design

Urban drainage entails the provision of protection from major and minor storms, which have quite different design approaches, as follows:

- *Minor Storms* - for minor storms, the design objective is usually to prevent flooding caused by frequent events (1:2 years < frequency < 1:10 years), which is sometimes termed ‘nuisance’ flooding. Runoff from minor storms is generally accommodated in a combination of surface and piped drainage systems, with the design frequency being subject to the preferences of the responsible authority.

- *Major Storms* - For major storms (1:50 years < frequency < 1:100 years), the design objective is to provide protection for life and property. In this case safe passage of runoff is generally achieved through a system of open channels, public collector roads and other public spaces (such as golf courses, parks and schools), both for conveyance of runoff, as well as detention.

Accommodation of minor storms is achieved with kerbs and channels drop inlets and underground reticulation. The runoff is initially collected in channels until the flooded width of the road reaches a specified limit and then discharged into the underground system, which is connected to an outfall point - typically a natural watercourse. In the case of high-speed routes, such as freeways, expressways and major arterials, no encroachment of stormwater onto the travelled lanes can be considered. Minor arterials and collectors should have one clear lane of 3.0 metres minimum width in each direction and local streets need only have one clear lane of 3.0 metres minimum width. In all cases, the 100-year storm should not cause a barrier kerb to be overtopped. The designer should ensure, therefore, that the gradients and crossfalls are sufficiently steep and the spacing of drop inlets sufficiently short to ensure that the recommended lateral spreads of water are not exceeded.

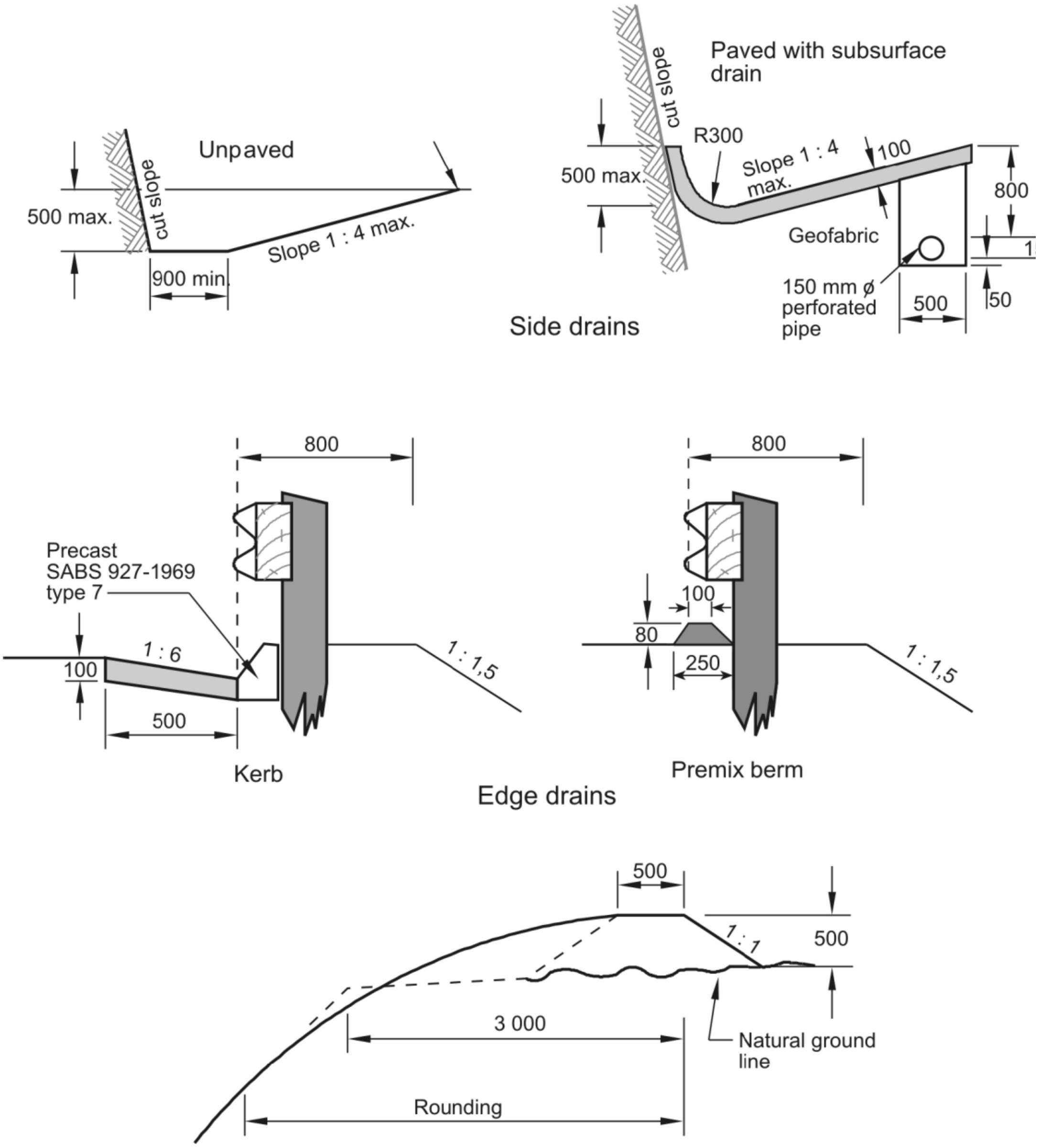

Figure 2.1: Typical Drain Types

2.4 Legal Aspects related to Water Infrastructure

2.4.1 Introduction

Stormwater is a component of the total water resources of an area and is covered by legal requirements relating to such resources. In planning, designing and constructing road drainage systems the relevant legislation and mandatory requirements relating to this resource need to be identified and complied with.

2.4.2 Water Resources Act

The Water Resources Act (Act No. 101 of 1993) covers the "optimum planning, development and use of Nigeria's water resources and other matters connected therewith". The Act charges the Minister of Water Resources with responsibilities relating to the protection of water resources, including the protection of "inland and estuarine fisheries, flora and fauna" and "the control and prevention of flooding, soil erosion and damage to watershed areas".

Section 9 of the Act covers the "unlawful diversion of water, etc." and reads as follows:

- As from the commencement of this Act, the diversion, storage, pumping or use on a commercial scale of any water or the construction, maintenance, operation, repair of any borehole or any hydraulic works shall be carried out only in accordance with a licence issued pursuant to this Act or regulations made thereunder.
- A person in breach of the provisions of subsection (1) of this section commits an offence under this Act.

The Act further states under Section 10 of the Act (Application for a Licence) that:

An application for the grant of a licence for the purposes mentioned in Section 9 of this Act shall be made to the Minister in such form and manner and shall contain or be accompanied by such information and documents as the Minister may, from time to time, prescribe.

Based on the above, it can be reasonably seen that any road construction activities that will have an impact on water resources (“rivers, creeks, streams, springs, lakes, lagoons, swamps, marshes or any other course for water in which water flows”, etc.) must comply with the provisions of the Act and that a licence may also be required.

2.4.3 Environmental Impact Assessment Act

The Environmental Impact Assessment Act (Act No. 86 of 1992) sets out “the general principles, procedure and methods to enable the prior consideration of environmental impact assessment on certain public or private projects”.

Section 2 of the Act states that, “the public or private sector of the economy shall not undertake or embark on or authorise projects or activities without prior consideration, at an early stage, of their environmental effects”.

The Act therefore covers not only drainage works, but also all road construction activities. Detailed guidance on the requirements of the Act for all road construction activities is provided in Volume VII (Environmental Management) of this manual (Highway Manual Part 1: Design) and therefore further guidance is not provided in Volume IV.

2.4.4 References

The South African Roads Agency Limited, 2013, Drainage Manual, 6th Edition.

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U.S. Department of Transportation, August 2001, Federal Highway Administration (FHWA), Urban Drainage Design Manual, 3rd Edition, Publication No. FHWA-NHI-10-009, Hydraulic Engineering Circular No. 22.

Ethiopian Roads Authority, 2001, Federal Democratic Republic of Ethiopia, Drainage Design Manual.

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Austroads, September 2008, (Austroads Publication No. AGRD05/08), Guide to Road Design, Part 5: Drainage Design.

UK Highways Agency, 1996-2009, Design Manual for Roads and Bridges (Various Advice Notes).

Government of Nigeria, 1992, Environmental Impact Assessment Act (accessed from http://www.placng.org/lawsofnigeria/ on 5 April 2013).

Government of Nigeria, 1993, Water Resources Act (accessed from http://www.placng.org/lawsofnigeria/ on 5 April 2013).

3: ECONOMICS OF DRAINAGE DESIGN

3.1 Introduction

The assessment of all investment in infrastructure ideally needs to be assessed to ensure the most economically efficient solution to the particular need is developed and implemented. However in certain situations there are either no alternative solutions to compare or the difference in investment costs and benefits are so small that the effort and cost in carrying out full economic analysis is not justified.

In the case of road drainage for most of the minor drainage and even some of the more major stream and river crossings, the solutions are either so specifically determined by the site and catchment conditions that there is only one real alternative, or the benefits of the various options are so similar that it reduces the economic analysis to a comparison of costs, to determine the most efficient solution.

In some cases however, usually relating to large river crossings or special site conditions, the generation of alternative options may be justified and then detailed economic assessment of the alternatives should be carried out.

As this Manual deals with the design of road drainage for highway engineers, the more general approach of cost considerations will be outlined, with only an overview of the necessary elements to be considered if a more specific economic analysis of options is required.

3.2 Cost Considerations

The design cost objectives is to minimize the total annual cost of the stormwater drainage facilities (capital costs, maintenance and operating costs, etc.). An associated objective is the reduction in average annual costs of damages by overflow or other aspects associated with lack of capacity in the system. Where overflows are evidences of incapacity of the storm system, investment to reduce the frequency of such overflows is more likely to be justified.

The basic factors making up the total costs of a drainage system are:

- Capital construction costs;
- Right-of-way or land acquisition costs;
- Effects of the improvement on property, particularly as to Federal liability;
- Traffic delays or diversion travel costs;
- Maintenance;
- Operation and administration; and
- Ultimate extension and/or replacement cost at the end of the useful life of the system.

The incremental cost between two different year flood frequencies shall be compared with the corresponding present worth of the annual increment of reduction in the cost of flood drainage so as to determine the culvert capacity.

Existing serviceable facilities including natural drainage swales, ditches, creeks, detention areas, etc. should be used wherever possible to reduce initial costs. For highways in urban areas, keeping the drainage facilities underground is often preferable to minimize the cost of land.

An overall consideration of optimum design of stormwater collection, storage and treatment facilities indicates that at least a balance should be struck among the capital costs, operation and maintenance costs, public convenience, environmental enhancement and other design objectives. Such an optimum balance is dynamic, changing over time with changing physical conditions and value perceptions.

3.3 Economic Evaluation

Where economic evaluation is justified, a comprehensive approach should be followed, taking into consideration all aspects of such evaluation. The elements to be considered are listed here for information but more comprehensive guidelines are provided in literature on economic evaluation.

3.3.1 Principles of Economic Evaluation

Economic evaluation attempts to determine the opportunity cost of using the resources required for the particular investment, as compared to the possible use of those resources in an alternative way. Governments normally set a target level of economic rate of return as a threshold to indicate the level of economic return at which investments are feasibile or viable in the country concerned. Above this threshold, alternative investments are compared against each other and the investment that provides the greatest economic return is then the more desirable option.

3.3.2 Economic Evaluation Techniques

There are several ways to compare the economic return of a particular option. Because benefits of an investment made at the present time are realised in the future, sometimes, only in several years' time, the value of these benefits has to be adjusted to take account of fact that the resources used could have been providing a return in the time in between. There are several approaches to this comparison, which will not be described in detail but will just be listed here for information:

- Present Worth of Cost (PWOC) technique
- Net Present Value (NPV) technique
- Benefit/Cost Ratio (B/C) technique
- Internal Rate of Return (IRR) technique

The economic value of resources is affected by supply and demand and by time and needs to take into account the following:

- Shadow pricing of the resources used to indicate the true value
- Future inflation
- The evaluation base date
- The time value of money
- The discounting procedure

3.3.3 Description of the Evaluation Process

The purpose of the evaluation of drainage systems and hydraulic structures is to assist in deciding the most effective and efficient option in which to invest. For major drainage systems and structures, large investments are required, which may vary considerably in terms of investment cost, delivery of benefit and in the timing of these benefits and costs. The whole procedure therefore includes the following steps:

- Establish the purpose of the economic evaluation
- Selection of appropriate economic evaluation technique(s)
- Definition of project alternatives
- Calculate the cost stream over the project life including, investment costs, operation and maintenance costs, the cost of abnormal floods and the risk of flood damage and the associated cost of diversion and delays
- Calculate the benefit stream over the project including, reduced operation and maintenance costs, reduction in vehicle running costs, time savings, reduction in accident cost, and avoidable costs of flood damage and the associated cost of diversion and delays; and
- Comparison of these by the economic evaluation technique selected

3.3.4 Timing of Project Implementation

Although a project may be viable and the most beneficial according to the comparison technique used, because the benefits accruing from that project may result later in the project life, it may mean that investment in that project can be delayed, with greater benefit. Where it is possible to delay the investment, this situation should be examined to determine the most efficient timing of the investment. This is usually done using the first year rate of return.

3.3.5 References

The South African Roads Agency Limited, 2013, Drainage Manual, 6th Edition.

U.S. Department of Transportation, August 2001, Federal Highway Administration (FHWA), Urban Drainage Design Manual, 3rd Edition, Publication No. FHWA-NHI-10-009, Hydraulic Engineering Circular No. 22.

UK Highways Agency, 1996-2009, Design Manual for Roads and Bridges (Various Advice Notes).

4: HYDROLOGICAL ANALYSIS

4.1 General

A hydrological analysis of the area to be drained is an essential element in the design of highway drainage. This type of study supplies the information on runoff and stream flow characteristics, which allow the calculation of the design flow that, is used as a basis of the hydraulic design of the system elements.

The design flow is dependent on:

- Characteristics of the storm or precipitation falling in the catchment.
- The response characteristics of the catchment area.
- The influence of temporal storage of the runoff within the catchment.

The design flow is established by selecting the proper combinations of these elements that can be reasonably expected to occur. This is usually further restricted by establishing an interval of time or frequency period as the basis of design. The design criteria would then be the maximum flow carried by the drainage structure with no flooding or a limited amount of flooding, to be exceeded on the average of once during a design period. Figure 4.1 shows a typical flood hydrograph illustrating the variation of discharge over time. For a given catchment the peak discharge (Q_p in this Figure) will occur when the duration of the maximum intensity storm is equal to the length of time taken for flood to reach its peak. This period is known as the time of concentration, T_c.

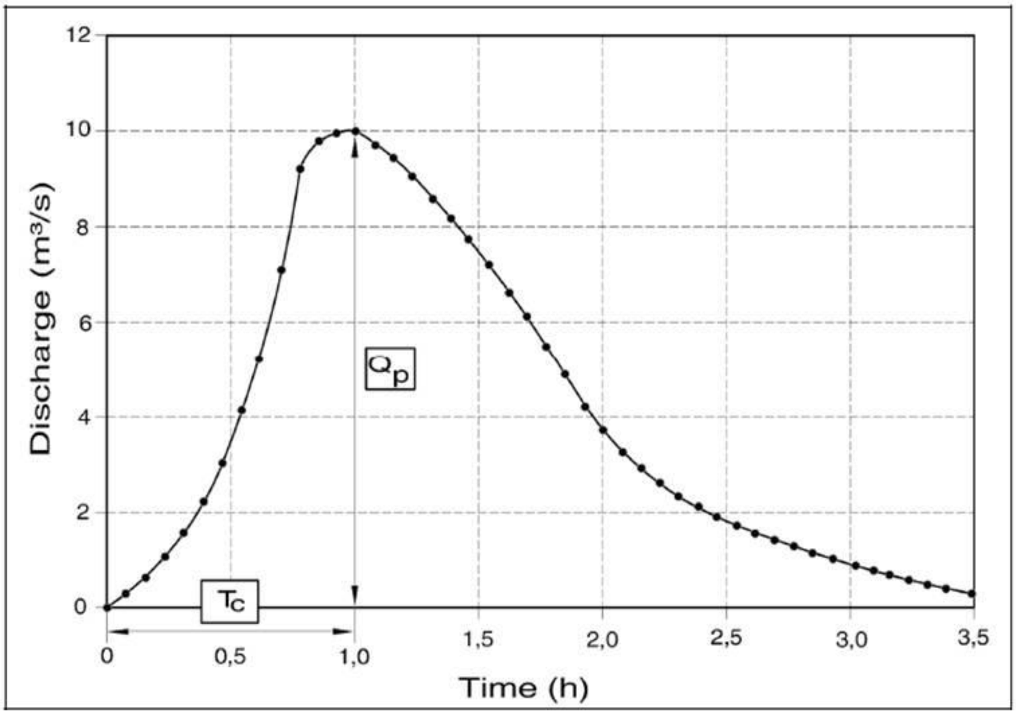

Figure 4.1: Typical Flood Hydrograph

4.2 Factors affecting Run-Off

The three factors main factors affecting run-off from a catchment are topographical, influence of development of the catchment and climatological factors:

4.2.1 Topographical factors

- Catchment size has a significant influence on run off, in a varying relationship as the size increases. For small catchments the peak discharge varies proportionally with catchment area (A), but for larger catchments the peak discharge becomes proportional to the square root of the area (√A).

- Catchment shape affects the peak discharge from such catchment, as rain falling on square (or round) shaped catchments may concentrate more quickly than in longer narrow catchments.

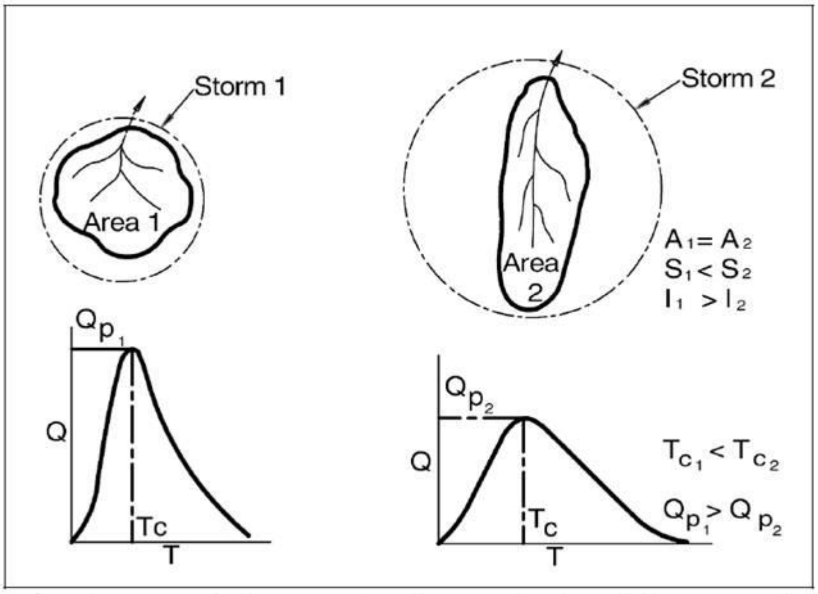

Figure 4.2: Effect of catchment shape on Run-off

- Catchment slope is important in determining peak discharge as steep slopes cause water to flow more swiftly and shorten the critical duration of the storm causing the flood.
- Stream patterns also affect the hydraulic efficiency of the waterways and meandering streams with many tributaries will result in lower peak discharge floods that more defined, well drained catchments.
- Infiltration of water into the soil of the catchment has an influence, which reduces as the soil becomes saturated. Soil types and geology will a have a significant influence on this aspect.
- Variations in vegetation will affect peak discharge, to the extent that there will be seasonal variations in the same catchment.
- Natural storage (dips and depressions, puddles and natural lakes) has the effect of reducing the peak discharge from a catchment.

4.2.2 Development influences

- Land use has a significant effect on run off and future development within the catchment area needs to be considered to assess likely future peak discharge from a catchment.
- The establishment of storage dams and or reservoirs may have an attenuating effect on peak discharge.

4.2.3 Climatological variables

- Rainfall (including hail) is the most important factor in affecting peak discharge from a catchment. Snow is not a factor in Nigeria.
- Type and time duration of rainfall in an area is also very significant, as run-off depends on the quantity, intensity, duration, size of storm, direction of travel and velocity of the rainfall event.
- Climate has an indirect effect on run-off through its effect on vegetation cover as indicated above.

4.3 Methods of Estimation of Design Flow

There are several methods used to calculate the design flow for road drainage systems, the use of which depends on the availability of appropriate data. These include:

- *Statistical methods*, where historical flood peak records exist, such as for rivers with established gauging stations (data should be sought from the Nigerian Department of Hydrological Services and the Nigerian Inland Water Authority).
- The *Rational Method*, which is a simplified method of relating the rainfall intensity and catchment characteristics, to obtain an estimate of the design flood. The application of this simplified method is considered to be limited to

catchments up to 12 km^2. For catchments of larger area adjustments are applied to prevent over estimation of the design flood.

- *Other deterministic methods* used around the world, such as the Unit Hydrograph Method and the US SCS Method – these methods generally require detailed calibration for a particular catchment or region and cannot therefore be readily used in Nigeria.
- *Empirical methods.*

The Rational Method is widely used and has formed the basis of most road drainage design previously and, with the use of adjustments for larger catchment areas, has been the basis of the design manual in the past. This method will be presented here, but other methods should be used in specific situations where appropriate historical and other data is available.

4.4 Rational Method of Estimating Peak Discharge

The runoff estimate or design flow depends on the duration and intensity of rainfall; storm frequency, size, slope, shape and imperviousness of the drainage area and the probable development of the area.

Run off estimates shall be based on the rational method, also. In this method the runoff is related to the rainfall intensity by the formula:

$$Q = 0.278CIA$$ **(Equation 4-1)**

Where:

Q = Quantity of runoff in cubic metres per second (m^3/s)

C = Coefficient of runoff expressed as a percentage of imperviousness of the watershed surface.

I = Intensity or rate of rainfall expressed in millimetres per hour for a certain time of concentration.

A = Area of the watershed in square kilometres.

The rational method is based on the following assumptions:

- The peak rate of runoff at any point is a direct function of the average rainfall intensity during the time of concentration to that point.
- The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.
- The time of concentration is the time required for the runoff to become established and flow from the most remote part of the drainage area to the point under design.
- The coefficient of runoff is the same for all storms of all recurrence probabilities

Although the basic principles of the rational method are applicable to large drainage areas, reported practice generally limits its use to urban areas of up to 12 km^2. For areas larger than this, storage and subsurface drainage flows result in an attenuation of the runoff hydrograph so that rates of flow tend to be overestimated by the rational formula method unless these are taken into account.

For watersheds with an area greater than 12km^2 a correction factor given by the formula below can be applied:

$$Area\ Reduction\ Factor = 1/e^{\left(1-\frac{12}{A}\right)} \qquad \textbf{(Equation 4-2)}$$

Where, A is the catchment area in km^2. However, care should be exercised in using the Rational Formula in such cases, as the above correction factor is a general approximation that does not take into account specific catchment characteristics.

An alternative to the above would be to divide the catchment into smaller sub-catchments, each less than 12 km^2, and to aggregate the peak flows from these catchments using hydrologic routing. However, this approach requires more complex analysis and special expertise, which is beyond the scope of a drainage design manual.

4.4.1 Runoff Coefficient

The runoff coefficient, C, is the most subjective variable of the Rational Method. The coefficient accounts for abstractions or losses between rainfall and runoff which may vary for a given drainage area as influenced by differing topographical, vegetation and climatological conditions. The selection of the appropriate value of C for a particular catchment requires inspection of the site and should be based upon experience and engineering judgement of the prevailing conditions in the catchment area.

To assist in establishing a value for C, typical values are shown in Table 4.2, with further guidance on permeability of common materials in Table 4.5. For rural areas, C is calculated by aggregating individual values for the catchment surface slope, catchment permeability and vegetation ($C = C_s + C_p + C_v$). In most cases it is desirable to develop a composite runoff coefficient based on the percentage of different surface types in the drainage area.

Table 4.1: Recommended Guide on the Value of C in Urban Areas

Land Use	Factor
Lawns	
Sandy, flat (<2%)	0.05 - 0.10
Sandy, steep (>7%)	0.15 - 0.20
Heavy soil flat (<2%)	0.13 - 0.17
Heavy Soil steep (>7%)	0.25 - 0.35
Residential Areas	
Houses	0.30 - 0.50
Flats	0.50 - 0.70

Land Use	Factor
Industry	
Light industry	0.50 - 0.80
Heavy industry	0.60 - 0.90
Business	
City centre	0.70 - 0.95
Suburban	0.50 - 0.70
Streets	0.70 - 0.95
Maximum flood	1.00

Table 4.2: Recommended Guide on the Value of C in Rural Areas ($C = C_s + C_P + C_v$)

		Mean Annual Rainfall (mm)		
Component	**Classification**	**<600**	**600-900**	**>900**
Surface slope (C_s)				
	Wet lands and swamps (<3%)	0.01	0.03	0.05
	Flat areas (3 to 10%)	0.06	0.08	0.11
	Hilly (10 to 30%)	0.12	0.16	0.20
	Steep Areas (>30%)	0.22	0.26	0.30
Permeability (C_p)				
	Very permeable	0.03	0.04	0.05
	Permeable	0.06	0.08	0.10
	Semi permeable	0.12	0.16	0.20
	Impermeable	0.21	0.26	0.30
Vegetation (C_v)				
	Thick bush and plantation	0.03	0.04	0.05
	Light bush and farm land	0.07	0.11	0.15
	Grassland	0.17	0.21	0.25
	No vegetation	0.26	0.28	0.30

Table 4.3: Guidance on permeability of catchment based on soil type

Material	Permeability rating
Gravel, coarse sand	Very permeable
Fine sand, sandy loam	Permeable
Silt, loam, clayey sand	Semi permeable
Clay, peat, rock	Impermeable

Another factor to consider in estimating the design flow is initial saturation. It is more likely that total saturation will occur in the case of a long return period event than in the case of a short return period event. To take this into account, it is recommended that an adjustment factor is applied to the coefficient to obtain a final C value, as shown in Table 4.4.

Table 4.4: Adjustment Factor for C due to Initial Saturation

Return Period Yrs.	2	5	10	20	50	100
Factor for steep and impermeable catchments	0.75	0.80	0.85	0.90	0.95	1.00
Factor for steep and impermeable catchments	0.50	0.55	0.60	0.67	0.83	1.00

Example:

Drainage area composed of 30% pavement and roofs, 50% cultivated land (sand and gravel) and 20% forests, rolling terrain.

Solution:

Pavement and Roofs	.90 x .30 = .27
Cultivated Land	.30 x .50 = .15
Forest	.15 x .20 = .03
Total	**.45**

For the conditions the runoff coefficient would be c =.45

In selecting runoff coefficients, anticipated future development of the area should be considered. The runoff factor selected should reflect the character of the area after the development. A minimum runoff coefficient in urban areas of 0.5 should be selected to cater for future development of the area, as well as saturation of the soil.

4.4.2 Time of Concentration

The *time of concentration*, t_c, at any point is the time required for runoff from the most remote portion of the drainage area to reach the point of interest. The most remote portion provides the longest time of concentration, but is not necessarily the most distant point in the drainage area. Since a basic assumption of the Rational Method is that all portions of the area are contributing runoff, the time of concentration is used as the storm duration in calculating the intensity. In calculating the time of concentration, distinction is made between overland flow and flow in defined watercourses.

a. *Calculation of the time of concentration for overland flow:*

This type of flow usually occurs in the upper reaches of catchments or in small flat catchments, where there is no clearly defined watercourse. Runoff is in the form of thin layers of water flowing slowly over the fairly uneven ground surface.

The Kerby Formula is recommended for the calculation of t_c in this case. It is only applicable where the slope is fairly even.

$$t_c = 0.604 \left(\frac{rL}{\sqrt{S}}\right)^{0.467}$$

(Equation 4-3)

Where,

t_c = time of concentration (hours)

r = roughness coefficient (
Table 4.5)

L = Hydraulic length of catchment measured along flow path from the catchment boundary to the point of interest or where flow enters a defined watercourse, whichever is shorter (km).

S = slope of the catchment (elevation difference divided by length of flow path) in m/m.

Table 4.5: Recommended values for overland flow roughness (r)

Surface description	Recommended value of 'r'
Paved areas	0.02
Clean compacted soil, no stones	0.1
Sparse grass over fairly rough surface	0.3
Medium grass cover	0.4
Thick grass cover	0.8

b. Calculation of time of concentration for a defined watercourse:

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service (now the National Resources Conservation Service, NRCS).

$$t_c = \left(\frac{0.87\,L^2}{1000\,S_{av}}\right)^{0.385}$$

(Equation 4-4)

Where,

t_c = time of concentration (hours)

L = Hydraulic length of catchment measured along flow path from the catchment boundary to the point of interest (km).

S = average slope of the catchment in m/m.

The average slope can be determined using a formula developed by the US Geological Survey (USGS) and referred to as the 10-85 slope method, as shown in Figure 4.3.

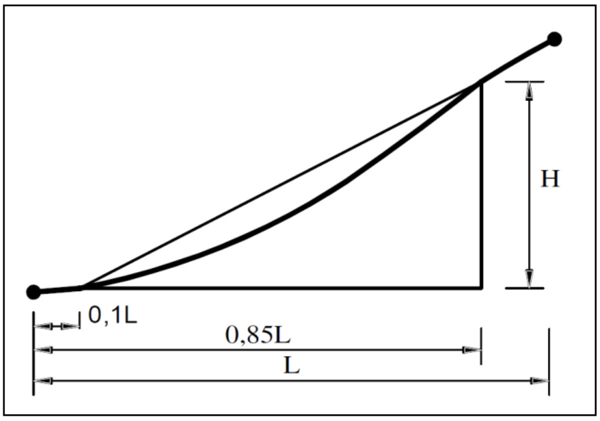

Figure 4.3: 10-85 Slope Method (USGS)

The formula below can be used for determining the 10-85 slope:

$$S_{av} = \frac{H_{0.85L} - H_{0.10L}}{(1000)(0.75L)}$$

(Equation 4-5)

Where:

s_{av} = Average slope (m/m).

$H_{0.10L}$ = Elevation height at 10% of the length of the watercourse measured upstream from the point where the peak flow is needed (m).

$H_{0.85L}$ = Elevation height at 85% of the length of the watercourse measured upstream from the point where the peak flow is needed (m).

L = Length of watercourse (km).

The height of waterfalls and steep rapids should be subtracted from the elevation difference used in the above calculation.

c. *Calculation of the time of concentration in urban areas:*

In urban areas the time of concentration should be determined where applicable by the means of estimated flow velocities calculated using the Manning equation for uniform flow through representative cross-sections with representative slopes.

A minimum time of concentration of 15 minutes is recommended for design except for inlets where a minimum of five minutes should be used.

4.4.3 Design Storm Frequency

Frequency in regards to hydraulic design is the average interval between discharges equal to or greater than a given discharge or the probability that such a discharge will occur in any one year. For example, a 10-year peak discharge is a flow that may be expected to be equalled or exceeded on an average of once every 10 years or 10 times in 100 years.

Selection of an appropriate design storm frequency or return period should be based on the degree and cost of repairing damage caused by exceeding the capacity of hydraulic structures combined with the hazards and inconvenience to the public.

Table 4.6 provides minimum recommended design frequencies for a range of drainage structures. Special additional considerations may be necessary in urban areas where the capacity of the receiving drainage system may dictate higher design storm frequencies than those specified above. Design frequencies for bridges and major culverts are provided in Volume V of this manual.

A summary of the four classes of road is provided below (refer to Section 2.1.3 of Volume 1 for a more comprehensive outline of road classification)

- *Class A: National Trunk Roads* – roads that link both nationally and internationally important centres, provincial capitals and main centres of population.
- *Class B: Primary Roads* - roads linking provincially important centres to each other or to a higher-class road (urban/rural centres).
- *Class C: Secondary Roads* - roads linking locally important centres to each other, to a more important centre, or to a higher-class road (rural/market centres).
- *Class D: Minor Roads* - any road linking to minor centres (market/local centre) and all other trafficable roads

Table 4.6: Design Storm Frequencies (years)

Type of Structure	**Class of Road**			
	A	**B**	**C**	**D**
Design of:				
Lesser/ Minor Culverts	50	20	10	10
Channel Changes	25*	25*	25*	25*
Storm Sewers	10	10	10	10
Side Drains	20	10	10	10
Stormwater Inlets	10	10	10	10
Gutters	10	10	10	10
Depressed Roadways	50	25	25	25
Checking of:				
Highway Embankment Overtopping	100*	50*	50*	50*
Excessive Damage due to Backwater	100*	100*	100*	100*

* Or largest flood on record.

4.4.4 Rainfall Intensity

Intensity is defined as the rate of rainfall and is given in the unit mm/hr. Although rainfall intensity varies during precipitation events, many procedures used to determine intensity assume a constant rainfall intensity to derive peak flow rates.

Determination of rainfall intensity for hydraulic design involves consideration of the following factors

- Average frequency of occurrence (also referred to as ‘return period’)
- Intensity duration characteristics of rainfall for selected average frequency of occurrence.
- Time of concentration.

There are various ways of determining the rainfall intensity; these are dependent on the rainfall records/data that is available.

Although the most widely used method is the one involving Intensity-Duration-Frequency (IDF) curves that have been developed for a specific location or region, two variations of this method are provided in this manual and are described in detail in the following sections. Method 1 is the more recent and preferred method.

The methods presented in this manual allow rainfall intensities to be determined for storm durations up to 24 hours. It is hoped that further research will result in the development of methods capable of accurately estimating design rainfall intensities for storm durations up to seven days. However, for most minor drainage structures (particularly those covered in this manual), it is likely that response times will be less than 24 hours. For larger structures, particularly bridges, response times exceeding 24 hours are not uncommon. For these structures, it is recommended that data be sought from a number of sources, including historical flow data from the Nigerian Department of Hydrological Services and the Nigerian Inland Water Authority; and rainfall data from the Nigerian Meteorological Services Agency (NIMET).

a. *Method 1 – Regional Method Developed by Oyebande (1983)*

The development of accurate methods for the estimation of design rainfall intensities for a particular city, region or country requires on-going measurement of rainfall using gauges that can measure the rate of rainfall or intensity. Currently, reliable data for the derivation of IDF curves across the whole of Nigeria is only available for the period prior to the early 1980s. The following method is a summary of a detailed report prepared by Professor Lekan Oyebande of the Department of Geography at the University of Lagos. The method allows for the calculation of the rainfall intensity at any location in Nigeria for a range of durations and storm frequencies or return periods.

Rainfall intensity-duration-frequency (IDF) studies were undertaken using annual extreme rainfall series available for 35 meteorological stations throughout Nigeria. In order to obtain data for design floods of high return period with a good level of confidence, the whole country was divided into ten rainfall zones. The IDF estimates were then used to generate curves for the ten different rainfall zones and for individual stations. IDF isohyetal maps for Nigeria were also generated from graphical estimates based on individual stations.

(i) Mathematical Method

Figure 4.4 below shows a map of Nigeria with the 10 principal rainfall zones with sub-zones defined by Oyebande (1983). The zones were designated according to the climatic and topographic characteristics shown in Table 4.6.

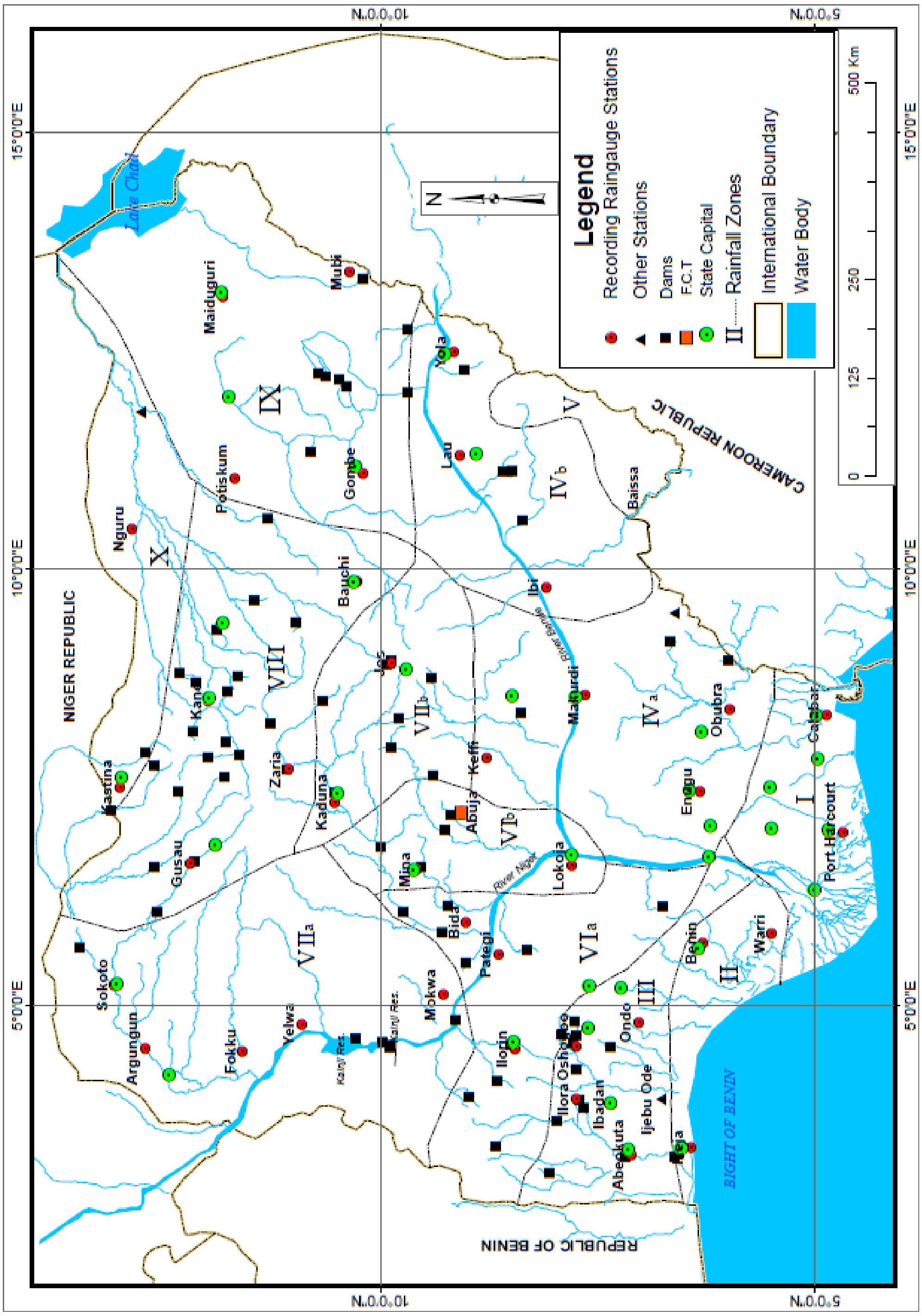

Figure 4.4: Rainfall zones of Nigeria with recording stations and Dams

Table 4.7: Ranges of characteristics of rainfall zones

Zone	A (%)	B (%)	C (%)	Altitude (m)	Annual mean rainfall (mm)	Annual mean no. of rainy days
I	41–46	47–90	43–49	80	2150	170
II	50–54	50–55	50–65	80	1550–2900	125–190
III	52–54	56–57	52–57	225–305	1217–1600	106–151
IVa	48–49	54–55	54–60	113–307	1224–1800	94–109
IVb	50–51	52–54	54–57	150–190	900–1150	75–90
V(a & b)	-	-	-	460–2409	1400–3670	-
VI	40–44	50–52	50–52	63–260	1190–1320	98–101
VIIa	59–67	59–67	53–66	119–351	710–1070	58–55
VIIb	59–63	63–67	56–61	645–1285	1218–1400	108–126
VIII	46–48	50–56	56–60	460–750	840–1085	50–82
IX	45–53	58–59	74–83	350–415	651–776	62–64
X	65–69	52–73	68–72	325–520	525–600	49–61

A = 100(10-year, 10-min. fall/10-year 30-min, fall);

B = 100(2-year, 15-min. fall/2-year 60-min, fall);

C = 100(25-year, 1-hr. fall/25-year 24-hr, fall).

There were no recording rain gauges in Zone V, the hilly eastern area adjacent to the border with the Cameroun Republic. It was therefore not possible to include this area in the method described in this Section, although intensity estimates for adjacent zones could be considered with caution in the absence of any other information.

Oyebande (1983) derived two basic equations for the calculation of rainfall intensity given the rainfall zone, return period and storm duration:

$$y = \alpha\,(x - \beta) \qquad \textbf{(Equation 4-6)}$$

$$y = \ln(T_r) - \frac{1}{(2T_r)} - \frac{1}{(24T_r^2)} - \frac{1}{(8T_r^3)} \qquad \textbf{(Equation 4-7)}$$

Where α and β are the scale and location parameters; x= rainfall intensity and T_r is the return period or frequency of occurrence. (Equation 4-6) becomes:

$$x = \beta + y(1/\alpha) \qquad \textbf{(Equation 4-8)}$$

The procedure for calculating the design rainfall intensity at a given location is as follows:

Step 1 – identify which Rainfall Zone the road or catchment falls within using Figure 4.2.

Step 2 – based on the storm duration or calculated time of concentration (Section 4.4.2), determine the values of (1/α) and β from Table 4.8 or Table 4.9 (Table 4.9 can be used where the road or catchment lies within or close to one of the cities shown). Linear interpolation may be required where the storm duration or time of concentration differs significantly from the available storm durations.

Step 3 – Using (Equation 4 – 6) and (Equation 4 – 5) and substituting the values obtained in Step 2 for (1/α) and β, as well as the Design Storm Frequency or Return Period (T_r), solve for x, the rainfall intensity in mm/hr.

The method is demonstrated in the following design examples:

Worked Example 1

Problem: Calculate a 25-year, 1 hour IDF for Lagos (Zone II).

Solution: For 1 hr. duration, $(1/\alpha)$= 14.36, β=55.06 (Table 4.8)

y_{25} (Tr =25 years) =ln 25-1/50 =3.22-.02 =3.20.

Thus, the 25-year rainfall intensity of 1hr duration = 55.06+14.36x3.2 =100.82 mm/hr.

Worked Example II

Problem: Calculate a 50-year, 3 hour IDF for Kano (Kano is located in Zone VIII)

Solution: For 3 hr. duration, $(1/\alpha)$= 6.05, β=15.37 (Table 4.9)

y_{50} (T_r =50 years) =ln 50-1/100 =3.91162 =3.911

Thus, the 50-year rainfall intensity of 3hr duration = 15.37+6.05x3.91162 = 39.035 mm/hr.

Table 4.8: Estimates of parameters of the equation $x = \beta + (1/\alpha)y$ for each zone

Zone	(1/α), β	0.2 h	0.4 h	1 h	3 h	6 h	12 h	24 h
I	1/α	23.52	19.19	12.10	6.63	3.74	1.87	0.98
	β	118.94	97.43	60.13	26.25	14.48	7.63	4.03
II	1/α	24.83	20.97	14.36	6.29	3.88	2.16	1.26
	β	113.16	85.74	55.06	23.58	13.30	7.03	3.81
III	1/α	19.57	14.99	11.32	5.59	2.94	1.49	0.85
	β	108.78	78.59	43.63	17.43	9.55	4.85	2.61
IVa	1/α	21.42	16.24	12.65	4.69	3.04	1.57	0.89
	β	111.83	83.38	49.13	20.03	10.56	5.61	2.83

IVb	1/α	20.15	15.87	12.70	6.30	3.56	1.84	0.92
	β	108.40	78.66	46.29	18.02	10.28	5.20	2.64
VI	1/α	32.95	21.76	13.49	6.03	3.13	2.26	0.57
	β	115.60	81.22	47.27	19.80	10.61	5.14	2.81
VIIa	1/α	19.40	15.62	9.21	3.52	1.60	0.88	0.39
	β	99.99	67.07	36.25	14.21	7.53	3.99	2.05
VIIb	1/α	23.28	17.95	9.83	3.87	3.13	1.14	0.68
	β	99.96	72.72	39.78	15.72	8.57	4.49	2.41
VIII	1/α	18.04	14.64	10.87	4.76	2.70	1.53	0.78
	β	95.65	69.79	40.67	15.92	8.46	4.47	2.28
IX	1/α	25.43	21.92	11.53	4.59	2.74	1.43	0.76
	β	98.23	73.30	40.32	15.85	8.65	4.39	2.23
X	1/α	21.53	17.65	9.35	3.86	2.09	1.08	0.57
	β	103.66	69.82	35.75	12.78	6.73	3.43	1.76

Table 4.9: Estimates of parameters of the equation x = β + (1/α)y for individual stations

Station	Zone	(1/α), β	0.2 h	0.4 h	1 h	3 h	6 h	12 h	24 h
Port Harcourt*	I	1/α	26.43	15.22	14.00	6.45	3.70	1.61	0.85
		β	121.03	100.13	61.35	27.31	15.53	8.10	4.29
Warri*	II	1/α	29.33	22.65	14.57	4.85	3.44	1.68	1.04
		β	120.78	92.47	59.91	27.53	15.06	7.90	4.52
Lagos*	II	1/α	23.55	22.17	16.66	6.82	3.93	1.97	1.11
		β	105.69	80.18	55.97	25.06	15.03	8.04	4.12
Ikeja*	II	1/α	23.32	18.35	10.99	4.69	2.94	1.62	1.00
		β	109.99	86.64	53.59	22.27	12.24	6.63	3.58
Oshogbo	III	1/α	26.47	12.89	7.51	5.27	2.73	1.25	0.76
		β	109.59	81.64	44.60	18.55	10.25	4.90	2.62
Enugu*	IVa	1/α	17.95	14.52	13.30	4.20	2.36	1.21	0.62
		β	121.05	92.27	53.34	22.00	11.85	6.12	3.26
Makurdi	IVa	1/α	27.13	20.54	11.61	4.54	2.57	1.39	0.73
		β	113.81	86.25	50.08	20.38	11.35	5.88	2.97
Iiorin*	IVb	1/α	18.85	12.19	10.30	3.73	3.15	1.56	0.95
		β	105.81	80.68	39.74	18.25	8.75	4.93	2.38
Yola	IVb	1/α	16.89	13.12	12.91	6.27	3.58	1.85	0.94
		β	102.12	73.83	44.53	16.96	9.69	4.90	2.49

Station	Zone	(1/α), β	0.2 h	0.4 h	1 h	3 h	6 h	12 h	24 h
Lokoja	VI	1/α	20.54	19.23	10.58	4.90	2.46	1.38	0.68
		β	101.17	82.31	47.22	18.95	9.93	4.92	2.57
Sokoto	VIIa	1/α	14.89	12.80	7.74	3.05	1.33	0.85	0.39
		β	90.11	60.26	32.92	12.75	6.80	3.76	2.00
Jos Aero*	VIIb	1/α	23.63	18.92	9.66	3.14	1.84	1.04	0.63
		β	101.64	70.32	39.41	14.85	7.98	4.21	2.28
Kaduma*	VIIb	1/α	19.82	15.04	10.00	4.50	2.34	1.24	0.72
		β	99.79	73.17	39.95	16.25	9.04	4.67	2.43
Kano	VIII	1/α	16.61	17.05	14.13	6.05	3.41	1.78	0.92
		β	90.64	66.62	39.87	15.37	8.12	4.13	2.11
Postiskum	IX	1/α	17.55	13.54	10.57	4.74	2.52	1.29	0.70
		β	90.76	71.00	41.43	15.92	8.47	4.28	2.17
Nguru	X	1/α	19.16	18.25	10.16	3.89	2.18	1.15	0.61
		β	106.34	70.46	35.11	11.97	6.30	3.25	1.66

*Record length of 20 – 29 years; other records are 14-19 years.

(ii) IDF Rainfall Maps of Nigeria

The estimates obtained from the family of curves in Appendix 1A were used for is arithmic mapping of the IDF estimates for selected durations and frequencies. The statistical surface generated is based on a kind of continuum concept. This continuum concept recognizes rainfall as a gradient of patterns in which values at points vary continuously in certain directions.

The statistical surfaces so specified provide a total view of the configuration of the statistical surface, or relative gradient, of extreme rainfall for each return period and duration represented (Oyebande, 1983).

The maps can be found in Appendix 1B and are based on individual stations which provided some 30 data/control points. This facilitated drawing of isohyets with greater reliability and accuracy. Such could not be achieved for the zones which would have provided only 10 data /control points for the whole of Nigeria. So by combining zones and station IDF data we have the best of two worlds. The frequencies and durations of rainfall depicted in the maps in Appendix 3 are presented in .

Table 4.10: Durations and Return periods used for the Rainfall IDF Maps of Nigeria (see Appendix 3).

Return period (frequency)	**Durations**			
2-yr	15 mins	30 mins	3 hrs	24 hrs
5-yr	15 mins	30 mins	1 hr	6 hrs
10-yr	15 mins	30 mins	1 hr	3 hrs
25-yr	15 mins	30 mins	1 hr	24 hrs
50-yr	-	-	1 hr	24 hrs

(iii) IDF Curves for Cities

Appendix 1A includes IDF charts for 35 rainfall stations throughout Nigeria. Data for these stations were used by Oyebande (1983) for the derivation of the mathematical method described in this section. These charts can be used as an alternative to the mathematical method where the road or catchment for which design flow estimates are required lies within one of the cities for which IDF curves are provided in Appendix 1A

b. Method 2 – IDF Equation (1973)

Method 2 was part of the original 1973 edition of the Highway Design Manual and is based on the work of the British West African Meteorological Services in Nigeria. Where IDF curves have not been developed values for the rainfall intensity are computed from the following formula:

$$I = \frac{K_n}{(t+a)^b} \qquad \textbf{(Equation 4-9)}$$

Where:

I = rainfall intensity in millimetres per hour.

t = time of concentration in hours

a & b = are station constants

$K_n = A + B \log_{10} n$

A & B =are station constants

n = storm frequency

The values for the constants a, b, A and B have been determined by the Meteorological Department. (See Meteorological Notes No 2 by the British West African Meteorological Services Nigeria for Lagos Area, Ikeja, and Kano.) The values for these station constants are shown in Table 4.11.

Table 4.11: Rainfall Station Constants

Station	Constant			
	a	b	A	B
Lagos (Apapa)	0.333	0.861	2.18	1.44
Kano	0.500	1.032	2.95	1.91
Ikeja	0.600	0.952	3.28	2.34

Rainfall records for other areas of Nigeria have not been used to determine other station constants. It is reasonable to assume the same constants for areas of similar rainfall regimes. In general Kano is subject to thunderstorm regime with their characteristic short sharp rainfalls rarely lasting more than three or four hours. Lagos Area on the other hand is subject the influence of monsoonal rains with generally low intensities and durations of up to 8 to 10 hours. At Ikeja for the greater part of the year the thunderstorm influence dominates but in the wettest months the monsoonal influence pre-dominates with intensities closer to those for Lagos. In areas of similar regimes the rainfall intensities would be proportional to the total rainfall. For example a locality with a similar storm regime as Lagos and a total annual rainfall twice that of Lagos would be expected to be subject to rainfalls of a given intensity on double the occasions. That is, the 5-year rainfall intensity for that location (for a given storm duration) would be the same as the 10-year rainfall intensity for Lagos. Intensity-duration-frequency curves for Lagos, Kano and Ikeja are provided in Appendix 1A.

4.4.5 Area

The total area of the catchment contributing to the peak discharge at a point under consideration must be determined. It may be measured accurately and is the only element of the rational method subject to precise determination. Boundaries of the drainage area may be established by field surveys or from suitable maps or aerial photographs.

Drainage area information should include also the following:

- *Land Use* – present and predicted future (including type and extent of vegetation cover), as it affects the degree of protection to be provided and percentage of imperviousness.
- *Soil* – character of soil and cover as they may affect the run-off coefficient.
- *Slope* - general magnitude of ground slopes, which with previous items and shape of drainage area will affect time of concentration.

4.5 Climate Change Considerations

Climate change is of growing concern to communities around the world, especially poorer communities that have limited capacity to respond or adapt to adverse impacts. One of the presumed consequences of climate change, which is particularly relevant to this design manual, is an expected global increase in the frequency and magnitude of rainfall events. Given this situation, it would be ideal if a designer could follow a simple procedure to estimate the likely increase in the design rainfall intensity at a given location over the lifetime of the respective drainage structure. However, current design approaches are generally based on the statistical analysis of historical data using the assumption that long term changes in design rainfall are negligible and therefore generally ignored. This makes consideration of expected future increases in rainfall intensity particularly difficult to quantify and take into account in the design process. In addition, despite a growing body of scientific knowledge and evidence, prediction of potential impacts on short duration rainfall intensity currently involves the downscaling of Global Climate Models (GCMs), which in itself is an approximate process where conflicting results are often obtained from different models.

Research by the Nigerian Meteorological Services (NIMET) generally indicates that the length of the wet season in Nigeria is now shorter than it was 30-40 years ago; while at the same time there has been a possible slight increase in mean annual rainfall. Accurate quantification of these impacts, however, remains particularly difficult and this situation is exacerbated by the naturally erratic nature of rainfall from year to year.

For the reasons given above, no method is prescribed in this manual for the quantification of climate change impacts. Historically, most failures of drainage structures have occurred as a result of either incorrect application of prescribed design techniques, poor construction or simply from the design flood frequency being exceeded. Addressing the first two issues above is therefore of primary importance in any engineering project. Where potential climate change impacts are of particular concern (possibly due to past events or failures), a design sensitivity analysis can be undertaken, whereby the ability of the selected drainage structure to withstand the design flow plus 20% can be evaluated and an educated decision on whether a larger capacity structure is required can then be made on the basis of this evaluation. In any drainage project, it is important that consideration is given to the potential implications arising from failure of a particular structure. While the recommended design flood frequencies provided in Table 4.6 should be used as a guide, if the impact of a failure is particularly high in terms of expected replacement cost and/ or disruption to traffic during redesign and construction, then consideration should be given to selection of a higher design storm frequency/ return period.

4.5.1 References

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5: HYDRAULIC ASSESSMENTS

5.1 Culverts

5.1.1 General

A culvert is a conduit used as an artificial channel under a roadway or embankment to maintain flow from a natural channel or drainage ditch. A properly designed culvert will carry the flow without causing damaging backwater, excessive constriction or excessive outlet velocities.

Designing highway culverts involves the determination of flow, hydraulic performance, economy, type of structure and location (position and spacing).

The engineering analysis shall also take into consideration foundation conditions, embankment construction, runoff conditions, soil characteristics, construction problems that may occur and any other factors that may be involved and are pertinent to the design and type selection.

5.1.2 Field Data

Information and field data required to properly design a culvert includes a description of the ground cover of the drainage area, a soil description of the stream bed at the proposed site, a topographic map showing contours and the outlining of the drainage area and for major culverts, a profile of the stream bed extending 150 metres upstream to 150 metres downstream from the proposed site with cross sections. The width of each cross section should extend beyond the limits of the flood plain on each side. Field surveys will be necessary where inadequate or no mapping exists. The proposed roadway alignment and culvert location must be shown. A detailed map to a scale not larger than 1:10 000 should be provided showing the roadway alignment and culvert location.

5.1.3 Culvert Hydraulics

Laboratory tests and field observation show two major types of culvert flow: (a) flow with inlet control and (b) flow with outlet control. For each type of control different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control the cross-sectional area of the culvert barrel; the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tail water in the outlet channel and the slope, roughness and length of the culvert barrel.

Determining the probable type of flow (inlet control or outlet control) under which a culvert will operate for a given set of conditions can readily be determined using freely downloadable public domain software, but a lengthy procedure is normally required for manual calculations. An alternative manual procedure that requires relatively simple calculations involves computing headwater depths from charts for both inlet control and outlet control and using the higher value to indicate the type of control and to determine the headwater depth. This method of determining the type of control is accurate except for a few cases, but the headwater in these instances is approximately the same for both types of control.

Both methods, following a manual calculation approach and using computer software, are valid design approaches. The choice of calculation method will therefore depend on the size of the culvert, tailwater restrictions (slope, channel capacity, etc.) and the preference of the designer. This manual outlines a manual calculation approach to culvert analysis, as this includes an explanation of the relevant theory. However, the computer software calculation method is generally a more accurate and faster method, although it is important for the designer to have a thorough understanding of the relevant theory.

For situations where downstream structures or impacts such as tide and/ or backwater are not relevant, the U.S. Federal Highway Administration (FHWA) HY-8 software can be used for culvert analysis, including inlet control, outlet control and the calculation of water surface profiles. The software will calculate the tailwater depth downstream of the culvert from the provided channel dimensions and slope and from this determine the flow regime and water surface profile in the culvert.

For situations where downstream structures, tides or other backwater effects influence water levels downstream of the culvert, backwater analysis using software, such as HEC-RAS (which can also undertake detailed culvert analysis) is recommended. A brief description of the capabilities of HY-8 and HEC-RAS is provided in Appendix 2.

Both inlet control and outlet control types of flow are discussed briefly in the following paragraphs and methods for the use of the charts are given.

a. Culverts Flowing with Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the entrance geometry, including the area, shape and type of inlet edge. Types of inlet controlled flow for a un-submerged and submerged entrance are shown in Figure 5.1A and B, respectively. A mitred (bevelled) entrance moves the control downstream to approximately the top of the mitre (see Figure 5.1C).

With inlet control the roughness and length of the culvert barrel and outlet conditions (including depth of tail water) are not factors in determining culvert capacity. The barrel slope has some effect on discharge but any adjustment for slope is considered minor and can be neglected for conventional culverts flowing with inlet control.

,The Division of Hydraulic Research, US Bureau of Public Roads (now the US Federal Highway Administration – FHWA), developed nomographs, based on research data for determining culvert capacity for inlet control. These nomographs give headwater discharge relationships for most conventional culverts with inlet control through a range of headwater depths or discharges.

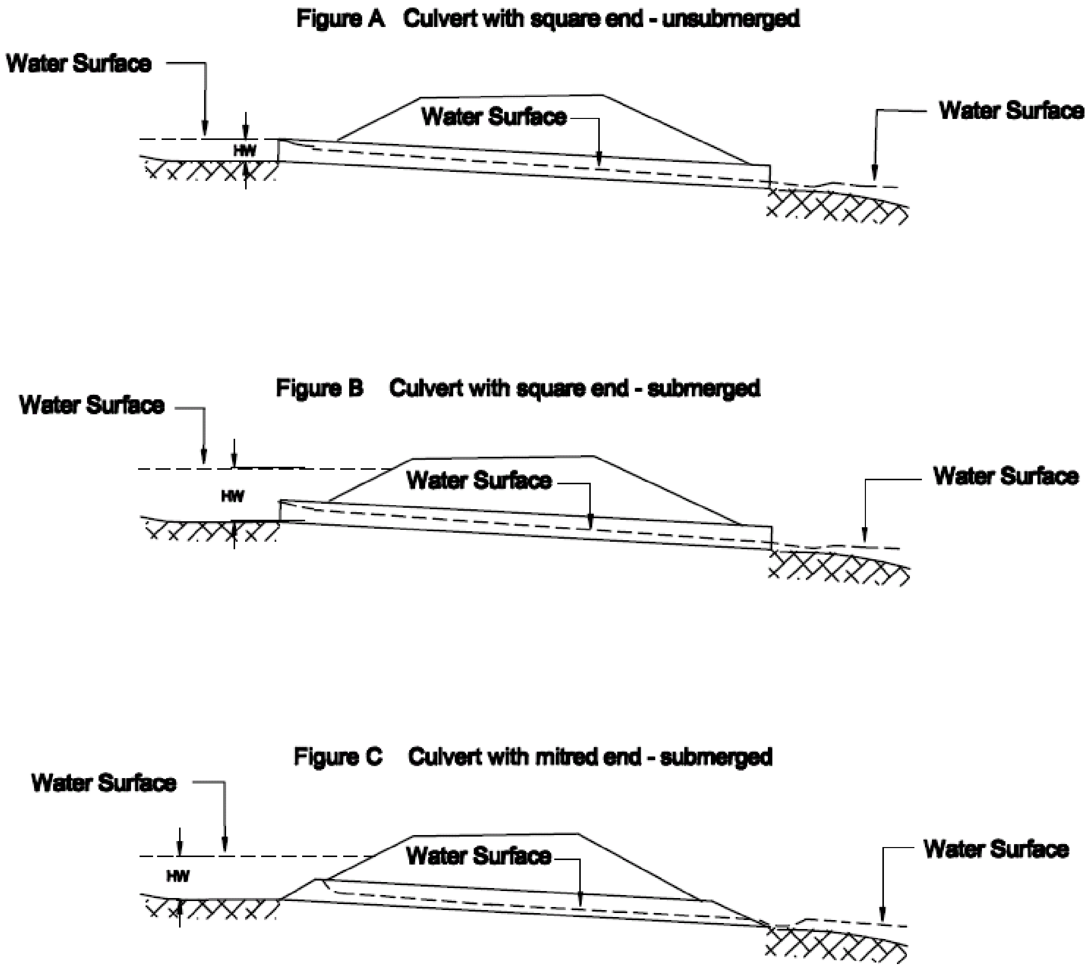


Figure 5.1: Culverts flowing under the Inlet Control condition

b. Culverts Flowing with Outlet Control.

Culverts flowing with outlet control can flow with the culvert barrel full for part of the barrel length or for all of it. If the entire cross section of the barrel is filled with water for the total length of the barrel the culvert is said to be in full flow or flowing full. Figure 5.2A and B illustrates these conditions. Figure 5.2C and D shows the other two common types of outlet control flow.

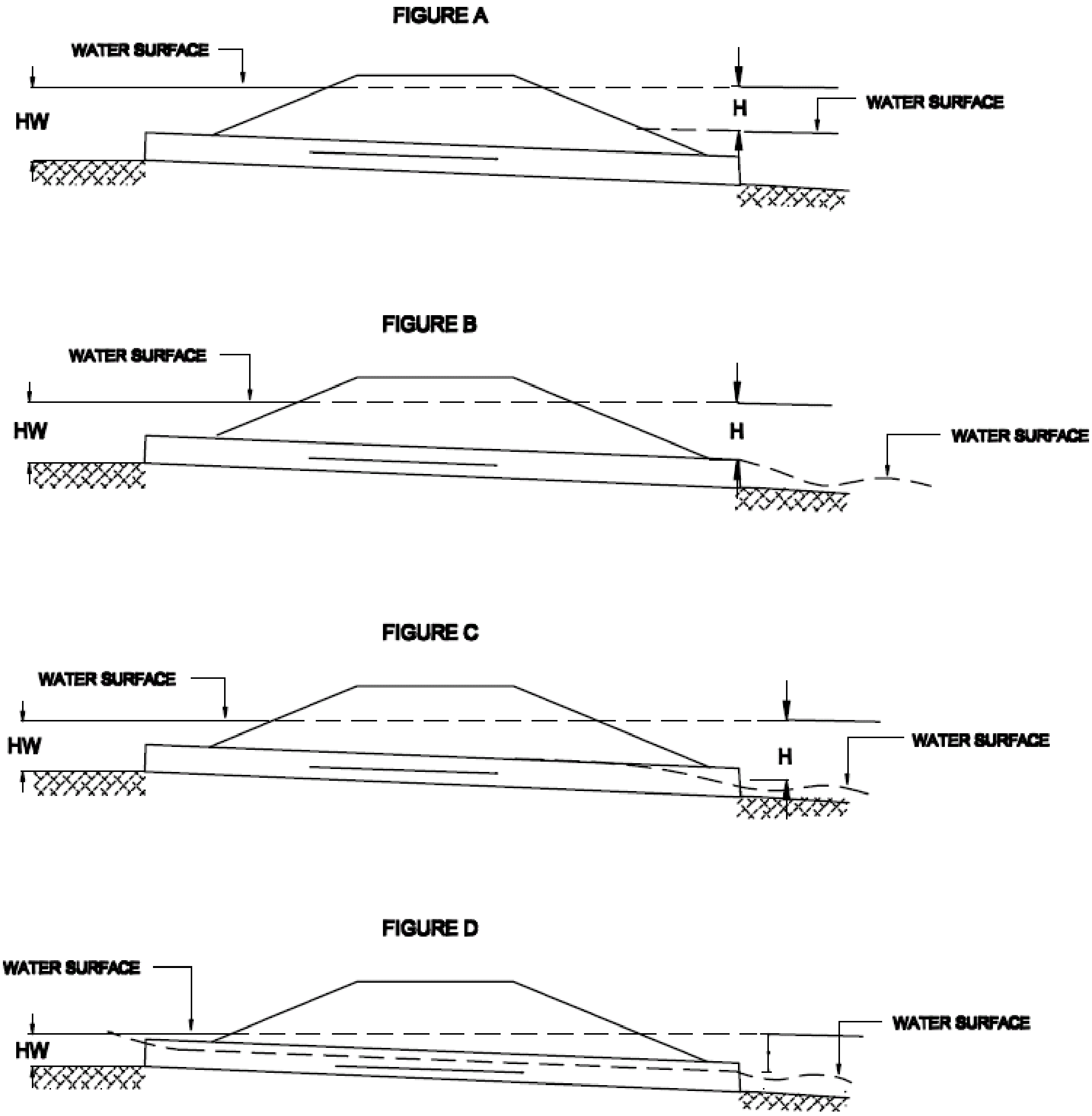


Figure 5.2: Culverts flowing under the Outlet Control condition

The manual procedure given in this manual for outlet control flow does not give an exact solution for a free water surface condition throughout the barrel length shown in Figure 5.2D. However, an approximate solution is given for this case when the headwater (HW) is 0.75D and above, where D is the height of the culvert barrel and an accurate solution can be obtained with computer software, such as HY-8.

The head H required to pass a given quantity of water through a culvert flowing in outlet control with the barrel flowing full throughout its length is made up of three major parts. These three parts are usually expressed in metres of water and include a velocity head H_v, an entrance loss H_e and a friction loss H_f. Expressed in equation form:

$$H = H_v + H_e + H_f$$ **(Equation 5-1)**

The velocity head H_v is obtained from the following relationship:

$$H_v = \frac{V^2}{2g}$$ **(Equation 5-2)**

V is the mean or average velocity in the culvert barrel in m/s and g is the acceleration due to gravity, 9.8 m/s^2

The mean velocity is found by dividing the discharge (Q) by the cross sectional area (A) of the flowing water. The velocity head is the kinetic energy of the water in the culvert barrel. This energy is obtained from ponding of the water at the entrance. Energy from the velocity of flow in the approach channel is neglected in this design procedure. Also, all of the velocity head (H_v) is assumed to be lost or, in other words, the exit loss coefficient equals 1.0.

The entrance loss (H_e) varies with the type or design of the culvert inlet. This loss is expressed in the following formula:

$$H_e = \frac{k_e V^2}{2g}$$ **(Equation 5-3)**

k_e is a coefficient given in Table 5.1, V is the mean or average velocity in the culvert barrel in m/s and *g* is the acceleration due to gravity, 9.8 m/s^2

The coefficient k_e for various types of culvert entrances is given in Table 5.1.

Table 5.1: Entrance Loss Coefficients

Type of Structure and Design Entrance	Coefficient k_e
Pipe Concrete	
Projecting from fill, socket end	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wings	
Socket end of pipe	0.2
Square edge	0.5
Rounded (radius = 1/12D)	0.1
Mitred to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Pipe, or Pipe Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wing walls	
Square edge	0.5
Mitred to confirm to fill slope	0.7
*End section conforming to fill slope	0.5
Box, Reinforced Concrete	
Headwall parallel to embankment (no wing walls)	
Square edge on 3 edges	0.5

Type of Structure and Design Entrance	Coefficient k_e
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wing walls at 30° to 75° to barrel	
Square edge at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension	0.2
Wing walls at 10° to 30° to barrel	
Square edge at crown	0.5
Wing walls parallel (extension of sides)	
Square edge at crown	0.7

The friction head, H_f, is the energy required to overcome the roughness of the culvert barrel. H_f, can be expressed in several ways. Since most Highway Engineers are familiar with Manning's n, the following expression is used:

$$H_f = \frac{19.6 n^2 L V^2}{R^{1.33} 2g}$$ **(Equation 5-4)**

Where

n = Manning's roughness coefficient (see Table 5.2)

L = Length of culvert barrel (m).

V = Mean velocity of flow in culvert barrel (m/sec).

g = Acceleration of gravity (9.8 IN/sec).

R = Hydraulic radius or the area of flow for full cross section (m^2) divided by the wetted perimeter of the culvert (m) A/WP

Rewriting equation 1 and simplifying, gives the following Equation 5 – 5 for full flow:

$$H = \left[1 + K_e + \underline{19.60n^2L} \right] \underline{V^2}$$

(Equation 5-5)

Equation 2 can be solved readily by the use of spreadsheet calculations or the full flow nomographs in Appendix 3. The equations shown on these nomographs are the same as Equation 5 – 6 expressed in a different form. Each nomograph is drawn for a single value of n as noted on the respective chart. These nomographs can be used for other values of n by modifying the culvert length as directed in the instructions for the use of the full flow nomographs.

Finding the value of H from a nomograph is not the complete solution for outlet control type of flow. Headwater must be determined and other factors such as slope of the culvert barrel and outlet conditions enter into this computation. The value of H in metres must be measured from some 'control' elevation at the outlet. This 'control' elevation is dependent on the rate of discharge or the elevation of the water surface of the tail water. For simplicity a value H_c is used as the distance in metres from the culvert invert (flow line) at the outlet to the 'control' elevation.

The following equation is used to compute headwater (HW):

$$HW = h_o + H - LS$$ **(Equation 5-6)**

Where S is the slope of the flow line in metres per metre, L is the length of the culvert in metres and H, h_0 and HW are all in metres. The determination of h_0 is discussed in the following paragraphs for the various flow conditions at the outlet.

If the water surface in the outlet channel (tail water elevation) is at or above the top of the barrel at the outlet as shown in Figure 5.3, A the solution for HW is simple. The depth of water in the outlet channel TW is equal to h_0 and HW can be established from the relationship to the other terms in Equation 5 – 6.

If the tail water elevation is below the top or crown of the culvert at the outlet the determination of h_0 for a given discharge and size of culvert is more difficult. Figure 5.2B, C and D are three common types of flow for outlet control with this low tail water condition.

In these cases h_0 is found by comparing two values

- TW depth in the outlet channel and
- $\frac{d_c + D}{2}$

h_0 is then set equal to the larger of these values.

The fraction, $\frac{d_c + D}{2}$, is a simplified means of computing h_0 when the tail water is low and the discharge does not fill the culvert barrel at the outlet. In this fraction d_c is critical depth as determined from Figure 12.14 through Figure 12.19 and D is the culvert height. The value of d_c should never exceed D, making the upper limit of this fraction equal to D.

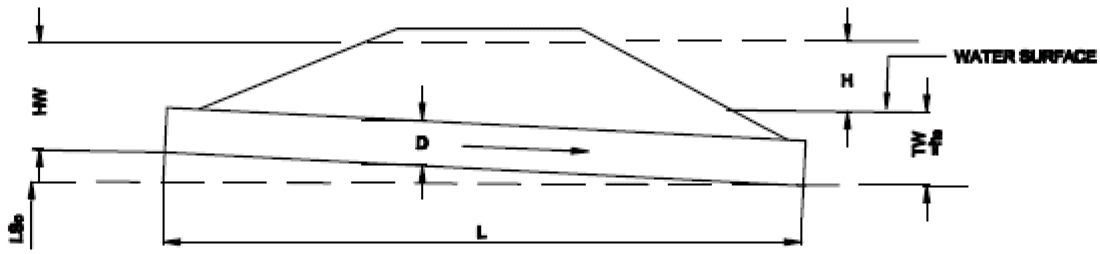


Figure A: Tailwater above culvert diameter

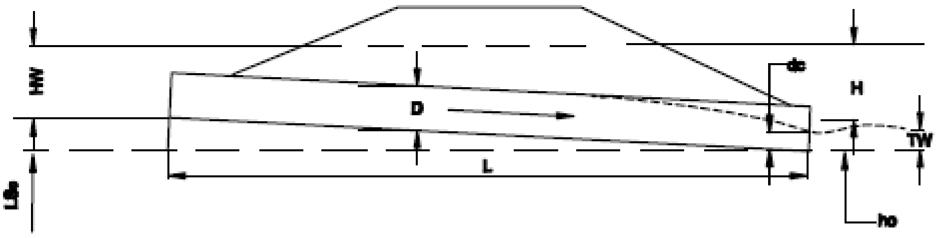


Figure B: Tailwater below culvert diameter

$$ho = \left\{ \frac{dc + D}{2} \text{ or } TW \right\}$$

Figure 5.3: Headwater Relationships for Culverts flowing under Outlet Control conditions

The sketch in Figure 5.3B shows the terms of Equation 5 – 6 for the cases discussed below.

From more rigorous solutions it has been found that Equation 5 – 6 gives accurate answers if the culvert flows full for a part of the barrel length as illustrated by Figure 5.3B. This condition of flow will exist if the backwater (as determined by Equation 5 – 6) is equal to or greater than the quantity:

$$D + (1 + k_e)\frac{V^2}{2g} \qquad \textbf{(Equation 5-7)}$$

Where V is the mean velocity for the full cross section of the barrel, k_e is the entrance loss coefficient, and D the culvert height. If the headwater drops below this point the water surface will be free throughout the culvert barrel as in Figure 5.2D and Equation 5 – 6 gives answers with some error as explained in the next paragraph.

In Figure 5.2D, Equation 5 – 6 is used to solve for HW when a free water surface exists through the barrel. Such a computation does not give a true value since the only correct way of finding HW in this case is by a backwater computation starting at the culvert outlet. However, Equation 5 – 6 will give answers of sufficient accuracy for design purposes if the headwater is limited to values greater than 0.75D. H is used in Figure 5.2D to show that the head loss here is an approximation of H. No solution is given for HW less than 0.75D.

c. Computing Depth of Tail Water

The depth of tail water is important in determining the hydraulic capacity of culverts flowing with outlet control. In many cases the downstream channel is of considerable width and the depth of water in this natural channel is less than the height of water in the outlet end of the culvert barrel, making the tail water ineffective as a control, so that its depth need not be computed to determine culver discharge capacity of headwater. There are instances, however, where the downstream water surface elevation is controlled by a downstream obstruction or backwater from another stream or culvert. A field inspection of all major culvert locations should be made to evaluate downstream controls and determine water stages.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation if the channel is reasonably uniform in cross section, slope and roughness. If the water surface in the outlet channel is established by downstream controls other factors must be found to determine the tail water elevation. Sometimes this necessitates a study of the stage discharge relationship of another stream into which the stream in question flows or the securing of data on reservoir elevations if a storage dam is involved.

d. Headwater

All culverts should be designed to carry the design frequency flood with a headwater depth that does not materially increase the size of the flooded upstream area over normal conditions. Allowable headwater depths are determined by the field conditions and are not associated with the design criteria. Therefore, the allowable headwater depths may vary from a few metres to considerable depths. It must be determined in the field what headwater depths can be permitted for the design flood and the 100 year flood, without causing excessive property damage.

A headwater depth of 1.25 times the culvert depth is recommended for cases where insufficient data is available to predict the flooding effect from the headwater depth.

5.1.4 Velocity

A culvert because of its hydraulic characteristics increases the velocity of flow above that of the natural channel. High velocities are most critical just downstream from the culvert outlet and the erosion potential from the energy of the water is a feature to be considered in culvert design.

The judgment of engineers working in a particular area is required to determine the need for energy dissipation at culvert outlets. All energy dissipaters add to the cost of a culvert and engineers should consider using them only when required to prevent a large scour hole or as remedial construction. As an aid in evaluating this need it is suggested that the outlet velocities be computed. The computed velocities can be compared with outlet velocities of other types and sizes of culverts and with the natural channel velocities. A change in size of culvert does not change outlet velocities appreciably in most cases.

Average outlet velocities for computing the normal velocity for the culvert cross may approximate culverts flowing with inlet control section using Manning’s equation:

$$V = \frac{1.486}{n} R^{2/3} S_0^{1/2}$$ **(Equation 5-8)**

Since the depth of flow is not known the use of tables or charts is recommended in solving this equation. The outlet velocity for inlet control computed in this manner will be high for culverts having a length-depth ratio less than say 20. For the shorter culverts velocities will be between those computed by Manning's equation and those occurring at critical depth. See 'Determination of Culvert Outlet Velocities", Section 5.1.20.

In outlet control the average outlet velocity will be the discharge divided by the cross-sectional area of flow at the outlet. This flow area will be between that corresponding to the critical depth and the full area of the pipe, depending upon the tail water conditions.

5.1.5 Waterway Area

It is generally desirable to maintain the natural stream width and depth by providing sufficient culvert waterway area at stream depth. This helps prevent excessive backwater or outlet velocities. This is accomplished by using multiple culverts, metal pipe arches, metal arches, and oval concrete pipe or box culverts (single or double).

5.1.6 Alignment

Generally culverts should be placed on the same alignment as the natural streambed to maintain the natural drainage system. However, pipes should have a straight alignment and straight entrance and outlet channels. This may require some modification of natural conditions. Bends and curves are permissible when extension of existing pipes requires a change in alignment or when straight alignment would require excessive and/or solid rock excavation. Bends and curves should be gradual to prevent excessive turbulence of flow and to allow debris to pass.

When natural conditions would require skewed alignments, the skew may be reduced or eliminated if necessary and the culvert shortened by using channel changes.

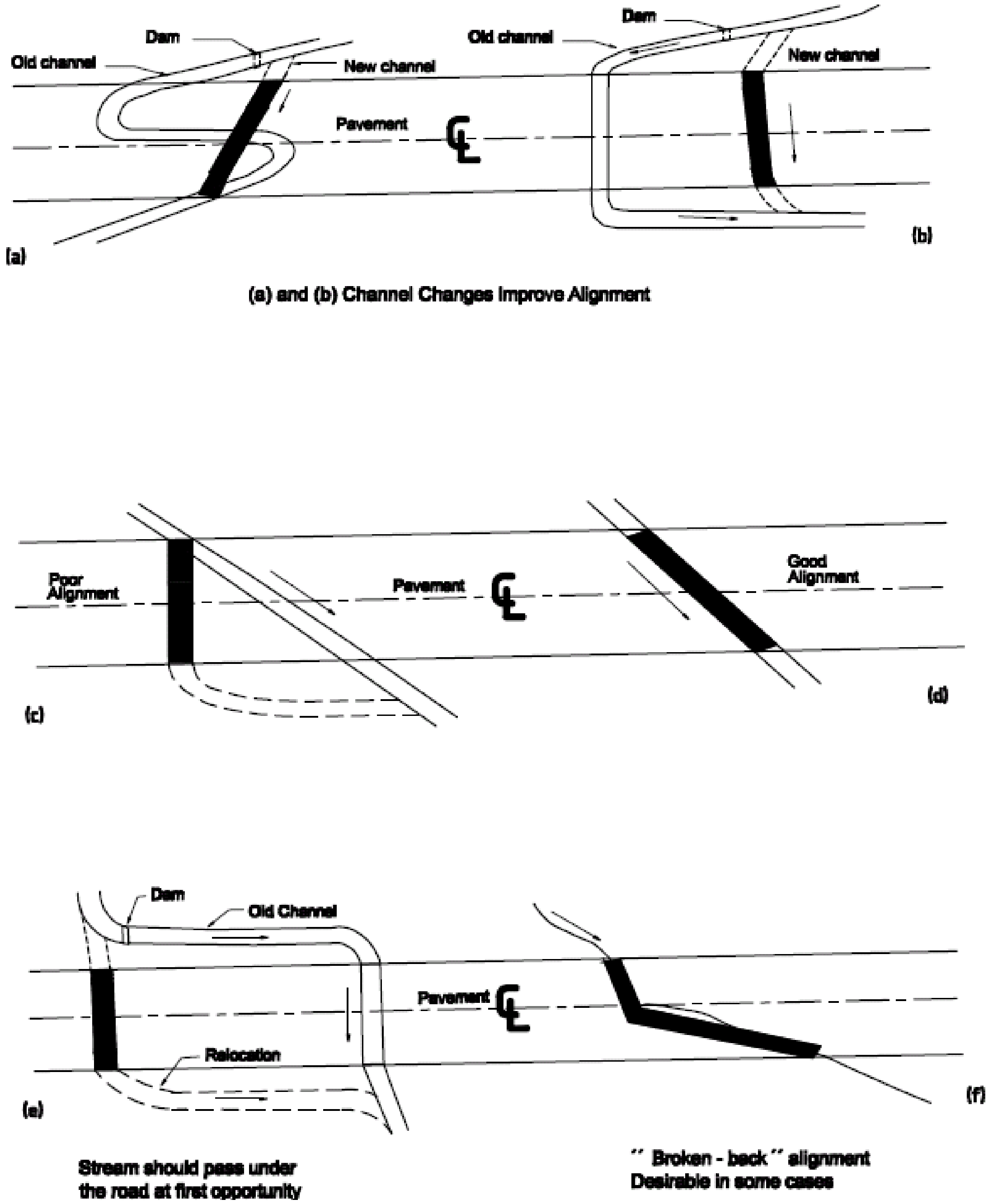


Figure 5.4: Alignment of Culverts

5.1.7 Grades

Most longitudinal culvert profiles should approximate the natural streambed. Other profiles may be chosen for either economic or hydraulic reasons. Modified culvert slopes, or slopes other than that of the natural stream, can be used to prevent stream degradation, minimise sedimentation, improve the hydraulic performance of the culvert, shorten the culvert, or reduce structural requirements. Modified slopes can also cause stream erosion and deposition. Slope alterations should, therefore, be given special attention to ensure that detrimental effects do not result from the change.

5.1.8 Requirements for Aquatic Organism Passage

In certain situations culvert designs will have special requirements to allow for the passage of aquatic organisms. Where such requirements are necessary, modification of the culvert slope from the natural streambed may not be feasible. It is typically more efficient to determine size requirements for aquatic organisms prior to undertaking any hydraulic calculations, as the former may dictate culvert size.

5.1.9 Culvert Type

The selection of the most appropriate type of culvert is dependent on a range of factors including economics, site conditions, and environmental considerations. While the majority of culverts consist of concrete pipe or box culverts, corrugated metal pipes, pipe-arch or arches, may be appropriate and economic in some situations. Box culverts are generally used where there is insufficient headroom for pipes or where the available waterway area for the culvert is at a minimum. In multi-cell construction, slab linked box culverts are often used for economy. Metal culverts have some advantages such as lower cost, and ease of transport and installation. However, they also have some serious disadvantages such as the potential for corrosion, damage due to poor construction or compaction, and higher cover requirements. Unless there are large financial savings, or other construction restraints, other more robust and more durable materials should generally be used.

5.1.10 Siltation/Blockage

The likelihood of blockage should be considered for all culverts. Blockage can occur through siltation or vegetation, though blockage by siltation is more likely to be temporary in nature. This is because during flood events, silt deposits can be removed by high velocity flows. To prevent siltation the desirable minimum velocity in the culvert should be above 0.7 m/s. A check of velocities should be undertaken as part of design. Where debris blockage is considered likely, larger culvert sizes may be required, in accordance with the extent of adverse impacts that could occur to the roadway or to surrounding properties. Blockage by debris is more likely to occur where the catchment contains significant woody riparian vegetation.

5.1.11 Allowable Grades

Concrete, steel and aluminium pipe may be used on all grades up to and including 20%. However, on grades ranging from 10% to 20%, concrete pipe anchors are required for concrete pipe that is furnished without the standard bell and spigot type ends. The function of the concrete pipe anchor is to hold the pipe in position. Normally, the pipe anchors are placed at 6 metre intervals, beginning with the first joint on the outlet end of the installation unless the site conditions dictate otherwise.

For grades over 20%, special anchoring devices are necessary for all types of pipe. The design depends on the grade, size of pipe and other considerations.

5.1.12 Minimum Size of Pipe

The minimum diameter of culvert pipes across a main roadway shall be 450 mm. Culvert pipes from graded inlets or catch basins in the roadway shall have a minimum diameter of 300 mm. Culvert pipes under driveways shall have a minimum diameter of 300 mm.

5.1.13 Structural Design

Culvert pipe shall be designed with limitations of fill heights and heights of cover as shown in Appendix 4.

The entire culvert length shall be designed for the maximum fill. This will provide for future widening of future lanes and eliminate the necessity of removing structurally inadequate culvert sections. For fill heights in excess of 23 metres, special designs will be required.

The height of overfill a culvert will safely sustain depends on the structural strength of and rigidity of the culvert barrel, foundation conditions and methods of installation, as described below:

a. Method “A”

All the backfill material shall be compacted. This method is applicable to both rigid and flexible culverts and it is usually the more economical method.

b. Method “B”

The pipe or box culvert shall be placed in a trench with vertical sides. Backfill material shall then be compacted to a level of 150 mm above the top of the culvert and loose granular backfill material placed in the succeeding layer above to a depth equal to the exterior vertical dimensions of the culvert barrel. This method is applicable to rigid pipes and box culverts. The loose material induces arch action in the soil, and this permits a higher safe overfill for the same strength of culvert than would be the case by Method “A”. Method B shall not be used for concrete pipes and box culverts where the depth of cover over the culvert is less than three times the structural depth of the barrel or box. The loose backfill shall terminate at a point on the side of the slope where the limiting height of loose cover is reached.

In all pipe culvert installations, the soil is required to exert passive pressures sufficient to support the pipe. When unsuitable material is encountered, granular material must be placed around the pipe in sufficient width to provide lateral restraint.

5.1.14 Bedding

A culvert pipe on unsuitable foundation material shall have gravel backfill for pipe bedding. Care must be exercised in all major installations to prevent too much seepage thought the embankment around the pipe. When a culvert is placed in embankments of silt or fine material the bedding and backfilling material must contain sufficient fines to prevent water flowing around and along the pipe and washing away embankment material. Cut-off walls or diaphragms may be used to help prevent seepage or flow along the pipe.

5.1.15 Limited Headroom

In low fills where a culvert depth or diameter is limited, multiple pipes, metal pipe, arches, metal arches, oval concrete pipe, or box culverts may be used. Minimum cover must be maintained on concrete or metal pipes. Metal and concrete pipes must have 600 mm cover except when placed under private road approaches or other minor intersections. Then the minimum height of cover shall be 300 mm. This minimum cover must exist over the length of pipe between the edges of shoulders.

5.1.16 Protective Treatment

All types of culverts are subjected to deterioration from corrosion or abrasion. Corrosion may result from active elements of the soil, water or the atmosphere. Mechanical wear depends on frequency, duration and velocity of the flow and the character and amount of bed load.

Protective treatment for steel culverts will have galvanizing and bituminous or polymer coating for the following situations:

- Where water is stagnant or where dense vegetation or leaves are likely to produce organic acids.
- Where lack of fall or an obstruction should result in deposition, continuous wetness or both.
- In locations of continuous flow.
- In well drained and normally dry alkali soils.

Under the following conditions bituminous coating of galvanized pipe usually does not give sufficient protection:

- Where excessive velocities are combined with abrasives in the flow.
- Where the culvert is subject to salt air or water.
- In highly mineralized soils, peat soils and alkaline soils that are poorly drained and frequently moist.
- Where industrial or farm waters are discharged untreated into the flow.

Concrete culvert pipe can be increased in durability by:

- Extra invert thickness may serve where erosion is likely to be so severe as to shorten appreciably the service life of the culvert. Extra wall thickness to provide additional cover over the reinforcing steel is warranted under exposure to corrosive environments and very high flow velocities.
- High-density concrete pipe achieved by spinning or other processes should be considered under exposure to salt air or salt water.

5.1.17 Multiple Pipe Spacing

When pipes are placed in a multiple installation the spacing between pipes shall be half the diameter of the pipes with a maximum spacing of 1.2 metres and a minimum spacing of 300 mm. The spacing shall be measured between outside surfaces of the pipes.

5.1.18 Culvert Joints

Rubber gaskets or hand mortared joints for concrete culvert pipe may be used: however rubber gasket joints should be called for when any of the following are applicable.

- Anticipated foundation conditions may be conducive to movement.
- Concrete pipe is used under fill heights of 4.5 metres or more.
- Concrete pipe is placed within embankment sections.

5.1.19 End Designs

a. *General*

Outlet and inlet structures are required for aesthetics, to prevent scouring of roadway embankments, to provide a transition from the culvert to the channel, and to improve the hydraulic performance of culverts. When end treatment is required, the type of end treatment shall be specified in the contract plans for each installation. For aesthetic reasons, end treatment should be considered for culvert ends that are exposed to view. This includes a view of the culverts from other than the roadway in urban and populated rural areas. Unless hydraulic or other considerations require special end treatments, the least expensive type of end treatment should be carried throughout the project for continuity and economy. It is desirable for safety considerations that ends of culverts be placed a minimum of 9.0 metres from the edge of the travelled way. This may be accomplished by warping the embankment in the area of installation. When it is not practical to extend the culvert end beyond the 9 metre recovery area, an end treatment or protective treatment such as guardrails should be considered in accordance with hazard evaluation procedures. The designer shall at all times be cognizant of the necessity of providing suitable alternates of both pipe and end treatment when alternates are feasible.

b. Projecting End

The projecting end is the simplest and most economical end design. This type of end provides no transition to help prevent scouring, therefore riprap may be required. When it is desirable to provide a transition to the channel, a headwall, header or a standard end section may be used, see Figure 5.5.

This type should be confined to concrete pipes since they are not subject to buoyancy failure and are inherently hydraulically efficient at the entrance. Metal pipes with a diameter of 750 mm or less may also have projecting ends.

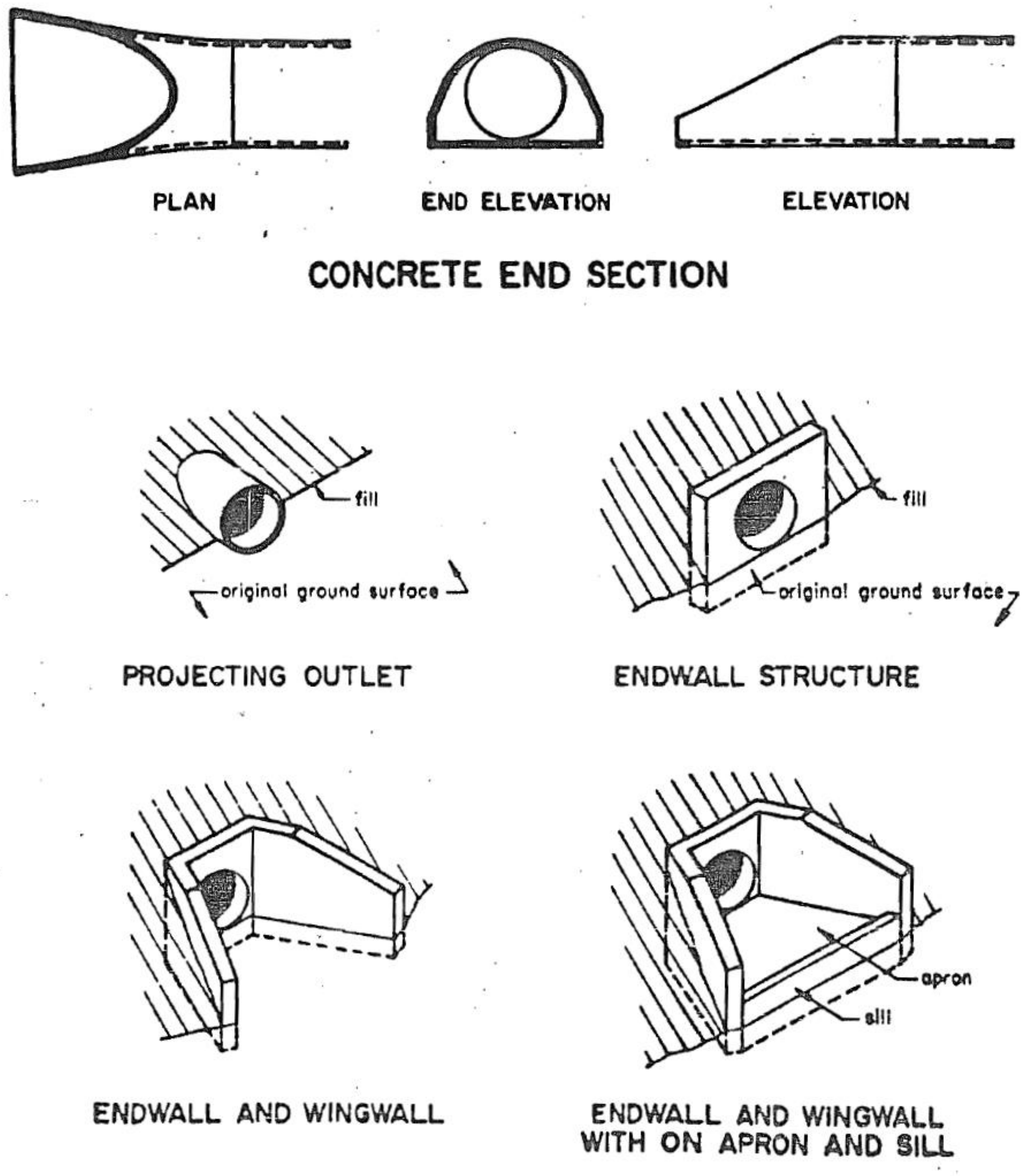


Figure 5.5: Types of Culvert End Treatment

c. Wing Walls

Wing walls retain and protect the embankment and provide a transition between the pipe and channel. They will normally consist of a concrete headwall and flared vertical wing walls, as shown in Figure 5.5.

The flare angle will normally be 15 degrees. The height and length must be sufficient to retain the embankment. Wing walls may be designed with or without an apron depending upon the ability of the stream bed to resist scour. Aprons must be provided with a cut off wall 600 mm below the invert to prevent scouring and undercutting. Wing walls may be constructed with a tapered 45 degree fillet for a more hydraulically efficient entrance.

d. End Sections

For culverts 750 mm in diameter and smaller, a bevelled end section is adequate for most installations and is the preferred treatment for aesthetics unless conditions require a higher type treatment. It is to be of the same material as the pipe to which it is attached.

The standard bevelled end section should not be used on pipes laid on a slew of more than 30 degrees from the perpendicular to the centreline of the highway. In these cases, special end treatment shall be provided if needed.

When required by hydraulic or erosive conditions, flared end sections may be used for pipes up to 2.1 metres in diameter and for pipe arches up to 2.125 m x 1.35 m. flared end sections for concrete culvert pipe may be either steel or aluminium. Whenever possible the contractor should be given the option of furnishing either steel or aluminium flared end sections for concrete pipe.

Flared end sections should be used in lieu of headwalls whenever feasible.

e. Headwalls

All metal pipes larger than 750 mm in diameter may be mitred at the entrance to conform to the fill slope. Mitring improves the hydraulics of the entrance and prevents buoyant uplift and failure of the pipe. Headwalls shall be used on mitred pipes at the entrance. Riprap may be used to prevent scour in addition to the headwall, but it does not prevent uplift and may aggravate abrasion by supplying material that may wash through the pipe.

f. Box Culverts

(i) Design

Single and multi-cell reinforced concrete box culverts shall be designed having regard to the live loads overfills method of overfill and method of bedding.

(ii) Inverts

Where low flows create a breeding place for mosquitoes a “V” bottom invert is recommended. When a culvert is to serve also as cattle pass a flat bottom shall be used.

(iii) Length

The length shall be such that the fill slope intersects the top of the culvert at the back of the headwall coping. The length may be shortened by increasing the headwall height and allowing the fill slope to intersect near the top of the headwall.

5.1.20 Example procedure for Design of Culverts

a. General

The culvert design should indicate the profile of the culvert flow line (invert), culvert length, allowable headwater depths for the design flood and the checking flood, roadway cross sections and roadway profile showing the height of fill. Hydraulic features of downstream controls tail water or backwater must be given.

b. Procedure for Selection of Culvert Sizes

Step 1: List given data

- Design discharge Q, in cubic metres per second.
- Approximate length of culvert in metres.
- Allowable headwater depth, in metres, which is the vertical distance from the culvert (flow line) at the entrance to the water surface elevation permissible in the approach channel upstream from the culvert.
- Type of culvert, including barrel material, barrel cross sectional shape and entrance type.
- Slope of culvert. (If grade is given in percentage, convert to slope in metres per metre).
- Allowable outlet velocity (if scour is a problem).

Note: It is suggested that culvert design sheets, similar to Figure 5.6 be used to record design data.

Step 2: Determine a trial size culvert.

- Refer to the appropriate inlet control nomograph (Appendix 3) for the culvert type selected.

- Using the allowable HW/D, determine HW or approximate this by using a HW of 1.25D for the entrance type to be used. Find a trial size culvert by following the instructions for use of these nomographs. If reasons for lesser or greater relative depth of headwater in a particular case should exist, another value of HW/D may be used for this trial selection.

- If the trial size for the culverts is obviously too large in dimensions because of limited height of embankment or availability of size, try a different HW/D value or multiple culverts by dividing the discharge equally for the number of culverts used. Raising the embankment height or the use of pipe arch and box culverts with a width greater than height should be considered. Selection should be based on an economic analysis.

Step 3 Find headwater HW depth for the trial size culvert.

- Determine and record headwater HW depth by use of the appropriate inlet control nomograph. Tail water TW conditions are to be neglected in this determination. HW in this case is found by simply multiplying HW/D obtained from the appropriate nomograph in Appendix 3.

- Compute and record HW for outlet control as instructed below:

  - Approximate the depth of tail water TW for the design flood condition in the outlet channel. The TW depth may also be due to backwater caused by another stream or some control downstream. An estimate of TW depth can be made by use of channel flow formulas or charts.

  - For tail water TW depths equal to or above the depth of the culvert at the outlet set TW equal to h_o and find HW by the following equation:

    $$HW = h_o + H - S_o L \quad \text{(Equation 5-9)}$$

    Where:

    HW = Vertical distance in metres from culvert invert (flow line) at entrance to pool surface upstream.

H = Head loss in metres as determined from the appropriate nomograph (Appendix 3)

h_o = Vertical distance in metres from culvert flow line at outlet to 'control' point. (In this case h_o equals TW).

S = Slope of barrel in m/m

L = Culvert length in metres

- For tail water TW elevations below the crown of the culvert at the outlet use the following equation to find headwater HW. It should be noted that this computation might contain approximations that are discussed under the heading 'Culverts Flowing with Outlet Control' in Section 5.1.3.

$$HW = h_o + H - SL \qquad \textbf{(Equation 5-10)}$$

Where:

h_o = $\dfrac{d_c + D}{2}$ or TW, whichever is the greater

d_c = critical depth in metres (Figure 12.14 to Figure 12.19).

D = culvert height in metres.

Other terms are as defined for Equation 5 – 8.

Note: When d_c exceeds D in a rectangular section h_o should be set equal to D.

- Compare the headwaters found in Step 3 (1) and Step 3 (2) (Inlet control or Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions.

- Compare the higher HW above with the allowable at the site. If HW is greater than the allowable repeat the procedure using a larger culvert. If the HW is less than the allowable, repeat the procedure to investigate the possibility of using a smaller size.

Step 4: Check outlet velocities for size selected.

- If outlet control governs in Step 3 above, outlet velocity equals Q/A, where A is the cross sectional area of flow at the outlet. If d_c or TW is less than the height of the culvert barrel use A corresponding to d_c or TW depth, whichever gives the greater area of flow.

- If inlet control governs in Step 3(3) above, outlet velocity can be assumed to equal normal velocity in open channel flow as computed by Manning's equation for the barrel size, roughness and slope of culvert selected. See Determination of Culvert Outlet Velocities, in Section 5.1.20 below.

Step 5: Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 6: Record final selection of culvert size, type, outlet velocity, required HW and economic justification.

c. Determination of Culvert Outlet Velocities

(i) Inlet Control

Using Manning's Equation to find velocity for full pipe,

Method:

- Determine R = D/4 where D = pipe diameter in metres.

- Lay a straight line on nomograph between slope and roughness to find a point on turning line.
- Lay straight line from R through turning point to find V.
- Compute Q for full pipe – Q_f = AV. A = 0.7854D^2.
- Compute (Q/Q_f) Q = Design Flow

Use Hydraulic Elements (Figure 10.1) to find V for design Q.

Method:

- Locate Q/Q_f along the bottom of the chart.
- Project vertically to intersect discharge curve.
- Project horizontally to intersect velocity curve
- Project downward to find V/V_f.
- Compute Design V = V_f (V/V_f)

(ii) Outlet Control

- When TW is at or above top of barrel

V = Q/A

Where A = area of pipe cross-section.

- 2. When TW is below top of barrel, find A for depth of flow for TW or d_c, whichever is greater. Then

V = Q/A.

Use Hydraulic Elements chart to find A.

Method:

- Compute d_c/D
- Locate d_c/D at side of chart
- Project horizontally to intersect area curve.
- Project downward to find A/A_f

Compute $A = A_f\ (\underline{A}/A_f)$

Note: Do not use Hydraulic Elements chart to find V directly. This chart is valid for uniform open channel flow conditions only. It does not apply when TW affects depth.

Table 5.2: Manning's Roughness Coefficients n

No	Conduit Type	Coefficient n
I	Closed Conduits	
	A Concrete pipe	0.011 - 0.013
	B Corrugated metal pipe arch:	
	1. 68 x 13 mm in corrugation (riveted pipe):	
	1.1 Plain or fully coated	0.024
	1.2 Paved invert (range values are for 25 and 50 percent of circumference paved):	
	a. Flow full depth	0.210 - 0.018
	b. Flow 0.8 depth	0.021 - 0.016
	c. Flow 0.6 depth	0.019 - 0.013
	2. 150 x 50 mm in corrugation (field bolted)	0.030
	C Vitrified clay pipe	0.012 - 0.014
	D Cast iron pipe, uncoated	0.013
	E Steel pipe	0.009 - 0.011
	F Brick	0.014 - 0.017
	G Monolithic concrete	
	1. Wood forms, rough	0.015 - 0.017
	2. Wood forms, smooth	0.012 - 0.014
	3. Steel forms	0.012 - 0.013
	H Cemented rubble masonry walls	

No	Conduit Type		Coefficient n
		1. Concrete floor and top	0.017 - 0.022
		2. Natural floor	0.019 - 0.025
	I	Laminated treated wood	0.015 - 0.017
	J	Vitrified clay liner plates	0.015
II	Open Channels, lined (straight alignment)		
	A	Concrete, with surfaces as indicated:	
		1. Formed, no finish	0.013 - 0.017
		2. Trowel finish	0.012 - 0.014
		3. Float finish	0.013 - 0.015
		4. Float finish, some gravel on bottom	0.015 - 0.017
		5. Gunite, good section	0.016 - 0.019
		6. Gunite, wavy section	0.018 - 0.022
	B	Concrete, bottom float furnished, sides as indicated:	
		1. Dressed stone in mortar	0.015 - 0.017
		2. Random stone in mortar	0.017 - 0.020
		3. Cement rubble masonry	0.020 - 0.25
		4. Cement rubble masonry, plastered	0.016 - 0.020
		5. Dry rubble (riprap)	0.020 - 0.030
	C	Gravel bottom, sides as indicated:	

No	Conduit Type	Coefficient n
	1. Formed concrete	0.017 - 0.020
	2. Random stone in mortar	0.020 - 0.023
	3. Dry rubble (riprap)	0.014 - 0.017
	D Brick	0.023 - 0.033
	E Asphalt	
	1. Smooth	0.013
	2. Rough	0.016
	F Wood, planned, clean	0.011 - 0.013
	G Concrete-lined excavated rock:	
	1. Good section	0.170 - 0.020
	2. Irregular section	0.022 - 0.027
III	Open Channels, excavated (straight alignment, natural lining)	
	A Earth, uniform section:	
	1. Clean, recently completed	0.016 - 0.018
	2. Clean, after weathering	0.018 - 0.020
	3. With short grass, few weeds	0.022 - 0.027
	4. In gravel soil, uniform section, clean	0.022 - 0.025
	B Earth, fairly uniform section:	

No	Conduit Type	Coefficient n
	1. No vegetation	0.022 - 0.025
	2. Grass, some weeds	0.025 - 0.030
	3. Dense weeds or aquatic plants in deep channels	0.030 - 0.035
	4. Sides clean, gravel bottom	0.025 - 0.030
	5. Sides clean, cobble bottom	0.030 - 0.040
	C Dragline excavated or dredged:	
	1. No vegetation	0.028 - 0.033
	2. Light brush on banks	0.035 - 0.050
	D Rock:	
	1. Based on design section	0.035
	2. Based on actual mean section:	
	a. Smooth and uniform	0.035 - 0.040
	b. Jagged and irregular	0.040 - 0.045
	E Channels not maintained, weeds and brush uncut:	
	1. Dense weeds, high as flow depth	0.080 - 0.120
	2. Clean bottom, brush on sides	0.050 - 0.080
	3. Clean bottom, brush on sides, highest stage of flow	0.070 - 0.110
	4. Dense brush, high stage	0.100 - 0.140
IV	Highway channels swales with maintained vegetation	

No	Conduit Type	Coefficient n
	(values shown are for velocities of 0.6 and 1.85 m/s)	
	A Depth of flow up to 215 mm	
	1. Bermuda grass, Kentucky bluegrass, buffalo grass	
	a. Mowed to 250 mm	0.070 - 0.045
	b. Length 100 – 150 mm	0.090 -0.050
	2. Good stand, any grass	
	a. Length about 300 mm	0.018 - 0.090
	b. Length about 600 mm	0.030 - 0.150
	3. Fair stand, any grass	
	a. Length about 300 mm	0.014 - 0.080
	b. Length about 600 mm	0.025 - 0.130
	B Depth of flow 215 – 1500 mm	
	1. Bermuda grass, Kentucky bluegrass, buffalo grass	
	a. Mowed to 50 mm	0.050 - 0.035
	b. Length 100 – 150 mm	0.060 - 0.040
	2. Good stand, any grass	
	a. Length about 300 mm	0.120 - 0.070
	b. Length about 600 mm	0.200 - 0.100
	3. Fair stand, any grass	
	a. Length about 300 mm	0.100 - 0.060

No	Conduit Type	Coefficient n
	b. Length about 600 mm	0.170 - 0.090
V	Street and express gutters	
	A Concrete gutter, troweled finish	0.012
	B Asphalt pavement	
	1. Smooth texture	0.013
	2. Rough texture	0.016
	C Concrete gutter with asphalt pavement	
	1. Smooth	0.013
	2. Rough	0.015
	D Concrete pavement	
	1. Float finish	0.014
	2. Broom finish	0.016
	E For gutters with small slope, where sediment may accumulate, increase above values of n by	0.002
VI	Natural Stream Channels	
	A Minor streams (surface width at flood stage less than 30 m)	
	1. Fairly regular section	
	a. Some grass and weeds, little or no brush	0.030 - 0.035

No	Conduit Type	Coefficient n
	b. Dense growth of weeds, depth of flow materially greater than weed height	0.035 - 0.050
	c. Some weeds, light brush on banks	0.035 - 0.050
	d. Some weeds, heavy brush on banks	0.050 - 0.070
	e. Some weeds, dense willows on banks	0.060 - 0.080
	f. For trees within channel, with branches submerged at high stage, increase all above values by	0.010 - 0.020
	2. Irregular sections, with pools, slight channel meander; increase values given above in VI, A.1.a – A.1.e.	0.010 - 0.020
	3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage	
	a. Bottom of gravel, cobbles, and few boulders	0.040 - 0.050
	b. Bottom of cobbles, with large boulders	0.050 - 0.070
	B Flood plains (adjacent to natural streams)	
	1. Pasture, no brush	
	a. Short grass	0.030 - 0.035
	b. High grass	0.035 - 0.050
	2. Cultivated areas	
	a. No crop	0.030 - 0.040
	b. Mature row crops	0.035 - 0.045

No	Conduit Type	Coefficient n
	c. Mature field crops	0.040 - 0.050
	3. Heavy weeds, scattered brush	0.050 - 0.070
	4. Light brush and trees	
	a. Winter	0.050 - 0.060
	b. Summer	0.060 - 0.080
	5. Medium to dense brush	
	a. Winter	0.070 - 0.110
	b. Summer	0.100 - 0.160
	6. Dense willows, summer, not bent over by current	0.050 - 0.020
	7. Cleared land with tree stumps 250 – 375 acre	
	a. No stouts	0.040 - 0.050
	b. With heavy growth of stouts	0.060 - 0.080
	8. Heavy stand of timber, a few down trees, little undergrowth	
	a. Flood depth below branches	0.100 - 0.120
	b. Flood depth reaches branches	0.120 - 0.160
	C Major streams (surface width at flood stage more than 30m): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Value of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028 - 0.033

CULVERT DESIGN FORM

PROJECT : ______________________

STATION : ______________ SHEET ______ OF ______

DESIGNER / DATE : ________ / ________

REVIEWER / DATE : ________ / ________

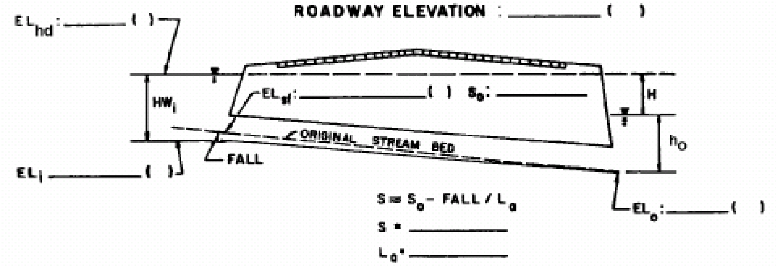
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: ______________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ______________
- ☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW ( )	TW ( )



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q ( )	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (K_u n^2 L)/R^{1.33}\right] v^2/2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS :

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE: ______________

SHAPE: ______________

MATERIAL: ________ n ________

ENTRANCE: ______________

Figure 5.6: Culvert Design Form

5.2 Storm Sewers

5.2.1 General

A stormwater sewer or stormwater drainage system is a system of drainage conduits which may have laterals, branches and trunks, and carries surface drainage or pavement runoff from catch basins or inlets, through manholes to any outfall. Any system with not more than 2 or 3 types of pipe and no change of pipe size may be classified as a culvert and be designed in accordance with Section 5.1 of this Volume of the Manual.

All storm sewer designs should be based on an engineering analysis and should take into consideration total drainage areas, runoff rates, pipe capacity, foundation conditions, soil characteristics, pipe strength, construction problems that may occur, and any other factors that may be involved and pertinent to the design.

5.2.2 Design Features

A storm sewer system must incorporate the following design features:

a. Soil Conditions

Surface and subsurface drainage must be provided to assure stable soil conditions necessary for adequate soil bearing capacity and protection of cuts or fills.

b. Inlet Spacing and Capacity

Refer to 6:.

c. Junction Spacing

Maintenance difficulties are minimized by providing catch basins or manholes at breaks in grade or alignment. Runs between catch basins should not exceed

100 metres for pipes smaller than 1200 mm and 150 metres for 1200 mm or larger pipes.

d. Future Expansion

If it is anticipated that a system may be expanded in the future provision for the capacity required by the expansion shall be incorporated into the design.

e. Velocities

Velocities of flow should be 1 metre/second or greater to prevent silting and clogging the pipes.

f. Silting Basins

Silting basins are used to prevent clogging or silting of sewer lines and should be incorporated with the use of cast metal outlets, concrete inlets, or any other inlet that does not have a silt basin.

g. Grades at Junctions

Crowns of pipe at the centre of manholes or catch basins should be at the same elevation. If a lateral must be placed so its flow is directed against the main flow through the manhole or catch basin, the lateral invert should be raised to match the crown of the inlet pipe.

h. Minimum Diameter Pipe

The minimum pipe diameter shall be 200 mm for runs 15 m or less and 300 mm for runs greater than 15 m.

i. Outfalls

Outfalls must be approved by all parties concerned. Storm runoff should not materially increase the flow of the receiving body. Erosion must be prevented by using headers, end walls, or riprap suitable bank protection. Installation of tide gates should be considered when the outfall is influenced by normal tidal action.

j. Location

Medians usually offer the most desirable storm sewer location. In the absence of meridians a location beyond the edge of pavement on the right of way or on casements should be used. It is generally recommended when a storm sewer is placed beyond the edge of the pavement that a trunk system with connecting laterals be used instead of running two trunks one down each side of the roadway. If a storm sewer must be located under the pavement, sufficient vertical clearance must be provided for making the proper inlet to storm sewer connections.

5.2.3 Sewer Hydraulics

Storm sewers are generally designed on open channel flow. In open channel flow the water surface is exposed to the atmosphere. Designing storm sewers to flow under pressure at peak flows can be advantageous since a smaller pipe size will be required than if open channel design were to be used.

The hydraulics of storm sewer design is based on Manning's formula:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \qquad \text{(Equation 5-11)}$$

Where:

Q = Design flow in cubic metres per second

N = Manning's coefficient of roughness

A = Area of pipe in square metres

R = Hydraulic radius

S = Slope of pipe in m/m

Manning's coefficients of roughness for pipes are shown on Table 5.2.

While design charts were generally used in the past for direct solution of Manning's equation, the now widespread use of computers allows engineers to solve for any of the variables in Manning's formula by a process of iteration. This can be undertaken by means of an iteration procedure such as the Newton-Raphson method, or by using the in-built functions available in most spreadsheet programs today, such as Solver in Microsoft Excel.

5.2.4 Sewer Design Data Required:

a. *Plans*

The design of a storm sewer system must begin with adequate field data. A map with a scale of 1:5 000 or 1:15 000 for urban areas and 1:30 000 for rural areas, must be provided showing the highway location through the area. The limits of the highway right of way should be shown to allow a check in excavation and embankment limits and sewer location encroachments. The location and elevations of existing or proposed utilities or structures that may be affected by or influence the location of the sewer must be shown. Profiles of the roadway, cross sections, horizontal curve data, super elevation diagrams, and paving and canalization plans showing kerbs, gutters, and ditches are required to establish drainage patters. Plans should indicate high and low points, crown and super elevation of the roadway by spot elevations or directional arrows.

b. Existing Drainage

If existing drainage structures are to be used, a map showing the existing drainage pattern must be provided. This map should show the extreme limits of the drainage area, the size, location and elevations of existing drainage facilities, the elevations of gutters, streets, lots, and other key points, and the location of any obstructions that might influence the manner in which the runoff pattern would be intercepted. The proposed methods of interception by the new facility may be shown on this map.

c. Drainage Map

The entire drainage system should be shown on a topographic map with suitable contour intervals. Since water will flow at right angles to contours, the drainage area may be plotted from the contours. These areas should be outlined or shaded and the pickup points indicated. Care must be taken to account for the influence of ditches, roads, and other features that may divert runoff from the natural runoff channels as shown by the contours. The drainage areas must be extended to the outermost possible limit regardless of how far from the roadway this may be unless it is otherwise provided for. When the drainage areas are too large to be adequately plotted on a map it will be permissible to record such an area and describe it with survey notes. Since the runoff from an area depends upon the ground cover and the type of soil, all portions of drainage areas must be classified.

d. Drainage System Plan

A map must be provided showing the drainage system, points of drainage interception, location of manholes, and location of outlets. If possible all necessary information should be shown on one map.

e. Elevations

Invert elevations, manhole top and bottom elevations, existing utilities, and ground line elevations should be shown on the roadway profiles or cross sections in the final contract plans.

5.2.5 Method of Computations

a. General

Storm sewer calculations should be shown on the “Storm Sewer Calculation” form (Figure 5.9). The form has five divisions: location, discharge, sewer design, sewer profile and remarks, which are further discussed below.

b. Location

The location division gives all the layout information of the sewer. Column 1 gives unique identifier or ID for the pipe. Column 2 gives a general location reference for the individual sewer lines, normally by the name of a street or survey reference line. Column 3 shows the stationing and offset of the inlets, catch basins or manholes, and Column 4 gives the length of pipe between the drainage structures given in Column 3.

c. Discharge

The “Discharge” division presents the runoff information and total flow into the sewer. Column 5 is used to designate the areas that contribute to the total drainage areas at a point in the sewer system. The areas should be numbered or lettered according to some reference system on the drainage area maps, used to lay out the system.

The type of ground cover (pavement, median, etc.) may be indicated, since drainage areas must be subdivided according to soil and ground cover types, a drainage area may have several different parts. Each part is to be given in km^2 to the nearest on hundredth. Column 6 shows the hectareage of the individual areas in km^2. Column 7 shows the assumed coefficient of runoff. Each increment of area must have a corresponding coefficient of run off. Column 8 is the product of Columns 6 and 7 giving the summation of CA for the area entering the drainage structure listed in Column 3. Column 9 lists the CA contributed form laterals. It should be noted that when laterals are added to a storm sewer they are included by adding CA factors (in Column 9) instead of the runoff total (in Column 14). The reason for this is that the time of concentration for the lateral will probably be different than the time of concentration for the sewer under design. Column 10 is the summation of Columns 7 and 8.

Column 11 gives the computed time of flow to a drainage structure to the nearest one tenth of a minute. When laterals are introduced into the system a check should be made to use the longest time of concentration at the junction. Column 12 is the time it takes the runoff (Column 14) to travel the length of pipe between drainage structures.

Column 13, rainfall intensity, is taken from the rainfall charts or the formula found in Section 4.4 of this manual. Column 14 is the product of Columns 10 and 13.

d. Sewer Design

This section represents the hydraulics of the sewer. The pipe slope, Column 17, is generally controlled by the ground line or drainage structures. Given the design flow and the slope of pipe, the size may be selected by using an iterative process of trying the pipe size in Column 15, with the slope in Column 16 and Manning's n in Column 15. The spread sheet then calculates depth in the pipe Column 21 and velocity Column 24. If the depth is between 0.7 and 0.9 of diameter D as shown in Columns 22 and 23, and the velocity is acceptable, then the pipe diameter can be used. If not that pipe diameter should be increased or decreased to the next standard size.

e. Sewer Profile

The invert elevations in Column 26 are the flow line elevations at the centre of the drainage structures. They should be set to provide a minimum of 600 mm of cover over the top of the pipe.

f. Hydraulic Gradient

This section is used as a check of the water elevation. The elevation of the hydraulic gradient at a drainage structure should never be higher than 300 mm below the elevation of the top of the inlet or manhole.

g. Remarks

The remarks section is useful in recording such information as design frequency, pipe roughness coefficients used or references to other design sheets.

5.3 Roadside Drainage Channels

5.3.1 General

Roadside drainage channels perform the vital action of diverting or removing surface water from the highway right of way. Gutters formed by kerbs, chutes, cut section ditches, toe of slope channels, intercepting channels, median swales and channel changes are all included in roadside drainage channels.

The design of a highway drainage channel to carry a given charge is accomplished in two parts. The first part of the design involves the computation of a channel section, which will carry the design discharge on the available slope. The second part of the design is the determination of the degree of protection required to prevent erosion in the channel.

A number of relevant publications have been developed by Highway and Roads Authorities in other countries, including References 1, 2, 8 and 9 in Appendix 5. In addition, Appendix 2 provides an outline of relevant public domain software that can be freely downloaded from the Internet, some of which can be used for the design of roadside drainage channels.

5.3.2 Hydraulics of Roadside Drainage Channels

In a drainage channel the quantity of water flowing (Q), the depth of flow (d) and the velocity of flow (V) depend upon the channel shape, roughness and slope (S).

The relationship between the above items is expressed in Manning's equation:

$$V = \frac{R^{2/3} S^{1/2}}{n} \qquad \text{(Equation 5-12)}$$

Where:

n = Manning's coefficient of channel roughness

V = Mean velocity in metres per second (m/s)

R = Hydraulic radius, in metres

S = Slope, in metres per metre

The values of the Manning's coefficient 'n' have been determined experimentally. The 'n' values for various types of channels are shown in Table 5.1.

R, the hydraulic radius is a shape factor that depends only upon the channel dimensions and the depth of flow. It is computed by the equation:

$$R = \frac{A}{WP} \qquad \textbf{(Equation 5-13)}$$

Where:

A = Cross sectional area of the flowing water in square metres taken at right angles to the direction of flow.

WP = Wetter perimeter or length, in metres, of the wetted contact between a stream of water and its containing channel measured in a plane at right angles to the direction of flow.

The capacity of the channel is determined from the equation:

$$Q = AV \qquad \text{(Equation 5-14)}$$

$$Q = \frac{AR^{2/3}S^{1/2}}{n}$$

Figure 5.10 is a nomograph for the solution of Manning's equation. "Design Charts for Open Channel Flow" published by the US Bureau of Public Roads contains charts for the solution of Manning's equation for rectangular, trapezoidal, and triangular channels.

5.3.3 Erosion Protection

Erosion control and maintenance can be minimized largely by the use of:

- Flat side slopes, rounded and blended with the natural terrain.
- Width, depth, slopes, alignment and protective treatment.
- Proper facilities for ground water interception.
- Dykes, berms and other protective devices.
- Protective ground covers and plantings.

The first step in providing for erosion protection is to check the actual velocity against the maximum safe values for the unprotected earth. When the velocity exceeds the maximum permissible, means for reducing velocity to safe levels or for protecting the channel should be used. Table 5.3 lists maximum permissible velocities for various erodible linings and Table 5.4 lists the maximum velocity for grass-lined channels.

Table 5.3: Permissible Velocities for Channels with erodible Linings, Based on Uniform Flow in Continuously Set, Aged Channels

Soil type or lining Water (Earth; no vegetation) (Earth; no vegetation)	Maximum Permissible Velocities for		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	m/s	m/s	m/s
Fine sand (non-colloidal)	0.4	0.7	0.4
Sandy loam (non-colloidal)	0.5	0.7	0.6
Silt loam (non-colloidal)	0.6	0.9	0.6
Ordinary firm loam	0.7	1	0.6
Volcanic ash	0.7	1	0.6
Fine gravel	0.7	1.5	1.1
Stiff clay (very colloidal)	1.1	1.5	0.9
Graded, loam to cobbles (non-colloidal)	1.1	1.5	1.5
Graded, silt to cobbles (colloidal)	1.2	1.6	1.5
Alluvial silts (non-colloidal)	0.6	1	0.6
Alluvial silts (colloidal)	1.1	1.5	0.9
Coarse gravel (non-colloidal)	1.2	1.8	2
Cobbles and shingles	1.5	1.6	2
Shales and hard pans	1.8	1.8	1.5

Table 5.4: Permissible Velocities for Channels lined with Uniform Strands or Various Grass Covers

Type of Lining	Allowable Velocity m/s
Well established grass on any good soil	1.8
Meadow type of grass with short, pliant blades, heavy strand, such as bluegrass	1.5
Bunch grasses, exposed soil between plants	0.6 – 1.2
Grains, stiff stemmed grasses that do not bend over under shallow flow	0.6 – 0.9

5.3.4 Alignment and Grade

The width of the right of way usually allows little choice in the alignment or in the grade of the channel, but insofar as practicable abrupt changes in alignment or in grade should be avoided. A sharp change in alignment presents a point of attack for the flowing water, and abrupt changes in grade cause deposition of transported material when the grade is flattened or scour when the grade is steepened.

A drainage channel should have a grade that produces velocities that neither erodes nor cause deposition in the channel. This optimum velocity also depends upon the size and shape of the channel, the quantity of water flowing, the material used to line the channel, and upon the nature of the soil and the type of sediment being transported by the stream.

Ordinarily the highway drainage channel must be located where it will best serve its intended purpose, using the grade and alignment obtainable at the location.

The point of discharge of a drainage channel into the natural watercourse requires particular attention. The alignment of the drainage channel should not cause eddies with attendant scum in the natural watercourse or near drainage structures. In erodible soils, if the flow line of the drainage channel is appreciably higher than that of the watercourse at the point of entry, a spillway or chute should be provide to discharge the water into the watercourse in order to prevent erosion in the drainage channel. The chute should be designed to prevent being undermined and destroyed.

5.3.5 Design Procedure

a. Layout of Drainage System

The layout of the drainage system should preferably be made on a topographic map which contains the location of the highway, the location of all drainage structures and the accentuated ridge and drainage lines. The edges of the right of way and of the roadway are drawn on the map. Then the drainage channels necessary to intercept the water before it reaches the roadbed are sketched in followed by the channels required to remove the water that cannot be intercepted before reaching the roadway. The quantities of water which must be removed when the design storm occurs are estimated for a few points along the drainage channels.

Recommended locations for computing incremental additions to the flow along the channels are locations where the grades change or the channel section is changed.

When the highway location cuts across tilled farmland, the highway drainage plan should be coordinated with the farm drainage system. Some farms are terraced and ploughed on contour lines. Thus the overland flow is collected and concentrated at points where the highway intersects the farm drainage channels, rather than occurring in a more or less uniform sheet over the hillside. Adequate notes by the locating party describing the farm drainage channels in the vicinity of the location centre line are essential to the design of facilities for handling the storm runoff.

b. Channel Grade

The approximate grade of the channel is computed from the topographic map. To prevent deposition of sediment he minimum gradient for earth and grass lined channels should be 0.5%. The channel grade should be kept constant or increasing in the downstream direction, insofar as practicable, to avoid deposition. The grade affects both the size of the channel required to carry a given flow and the velocity at which the flow occurs.

c. Channel Alignment

Changes in channel alignment should be as gradual as the width of right of way and terrain permits. Whenever practicable, changes in alignment should be made in the reaches of the channel which have the flatter slopes.

d. Channel Section

In general open channels adjacent to the roadway should have a section with side slopes not steeper than 4:1 (horizontal to vertical) and rounded bottom at least 1.2 metres wide. The rounding (vertical curves) at the junctions of the side slopes and the bottom do not appreciably affect the capacity, but they do add greatly to the safety and appearance of the channel. The depth of the channel need only be deep enough to carry the flow with a freeboard between 90 mm and 150 mm on flat slopes, except where subsoil drainage requires a deeper channel.

Figure 5.7 and Figure 5.8 provide guidelines on the foreslopes and backslopes that will be traversable (the shaded areas) and are preferable. Serious safety problems can be expected if the channel section falls outside the shaded area and is located where high-angle encroachments (outside of relatively sharp curves) are expected.

In Figure 5.7 it can be seen that, for a front slope of 1:4, the back slope should not be steeper than 1:6.5 (for channels with abrupt slope changes). From Figure 5.8, it can be calculated that the back slope should be flatter than 1:3.75 for a front slope of 1:4 (for channels with gradual slope changes).

e. Channel Capacity

Generally the discharge to be carried is estimated for several points along the channel. The points selected should include a section immediately above sharp breaks in grade and the points of entry of concentrated flow. A trial size of channel must be selected and the depth of the channel required to carry the flow computed. The trial size is adjusted until a size is found that will carry the design discharge and the need for protection is determined, freeboard is added to the required depth.

When a standard channel section has been adopted as a minimum section, a capacity table for various grades and types of lining could be prepared as a guide to the adequacy of the standard channel section for a particular site.

The capacity of the trial channel can be increased by increasing the grade the bottom width, the depth, or by decreasing the resistance of the channel through the use of a smoother lining. Increasing the bottom width has the least effect on the velocity of the flow and is generally the desirable way to increase capacity for a given depth and types of channel when the velocity is near the permissible limit.

This chart is applicable to all vee drains (ditches), rounded channels with bottom width less than 2.4 metres and trapezoidal channels with bottom widths less than 1.2 metres.

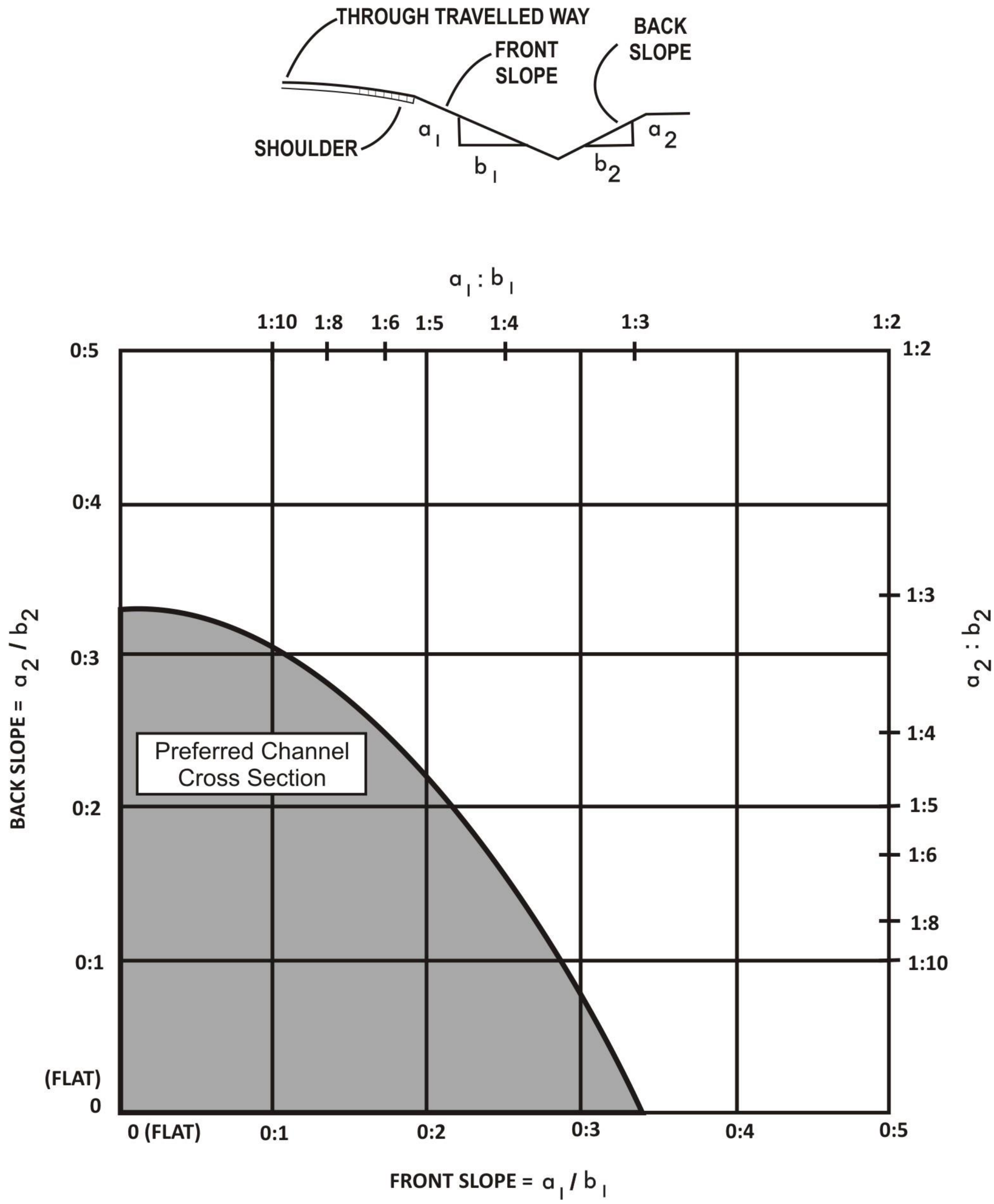


Figure 5.7: Preferred cross-sections for channels with abrupt slope changes

This chart is applicable to rounded channels with a bottom width of 2.4 metres or more and to trapezoidal channels with bottom widths equal to or greater than 1.2 metres

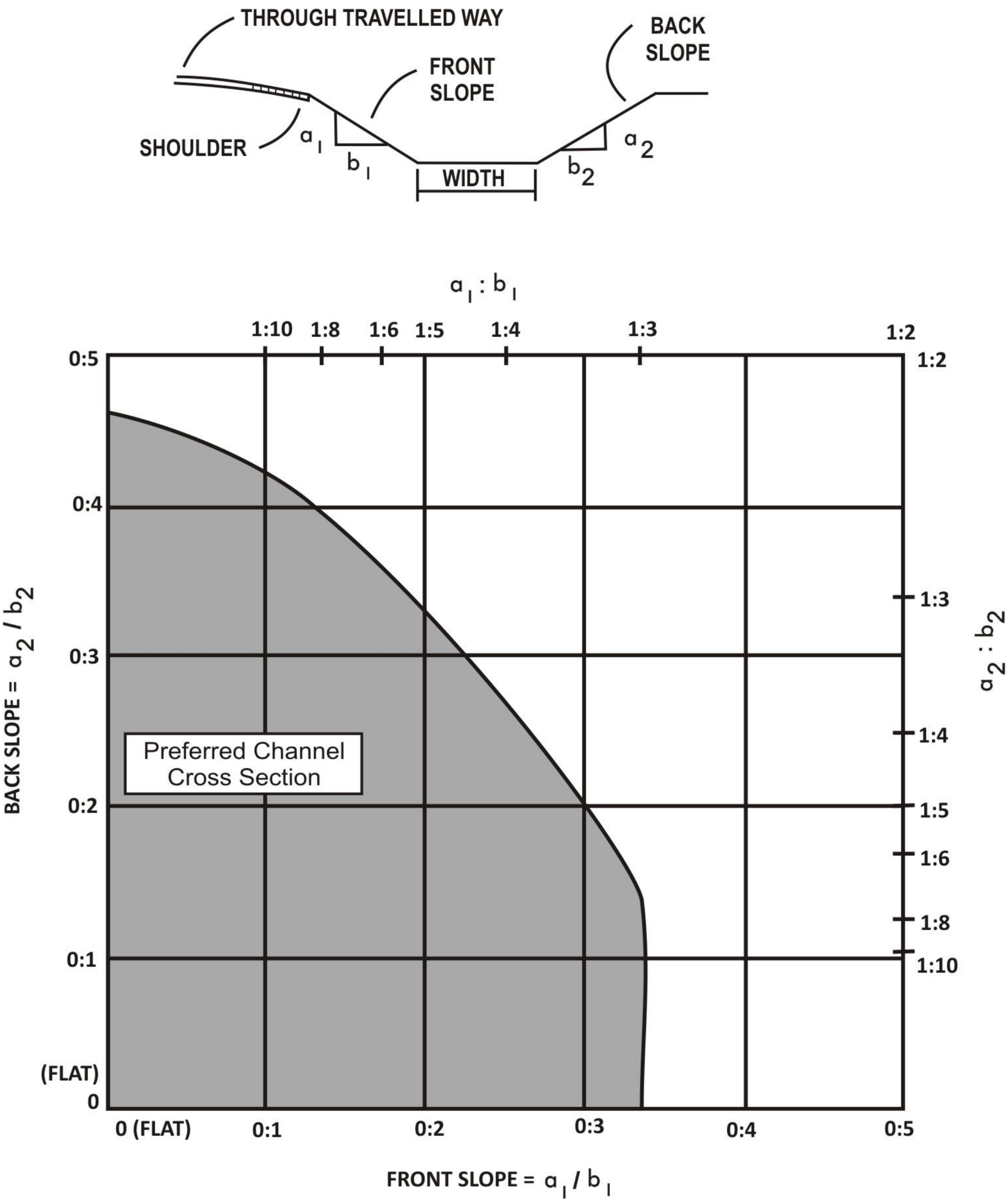


Figure 5.8: Preferred cross-sections for channels with gradual slope changes

Pipe Sizing Calculations

Date: *11/11/2008*

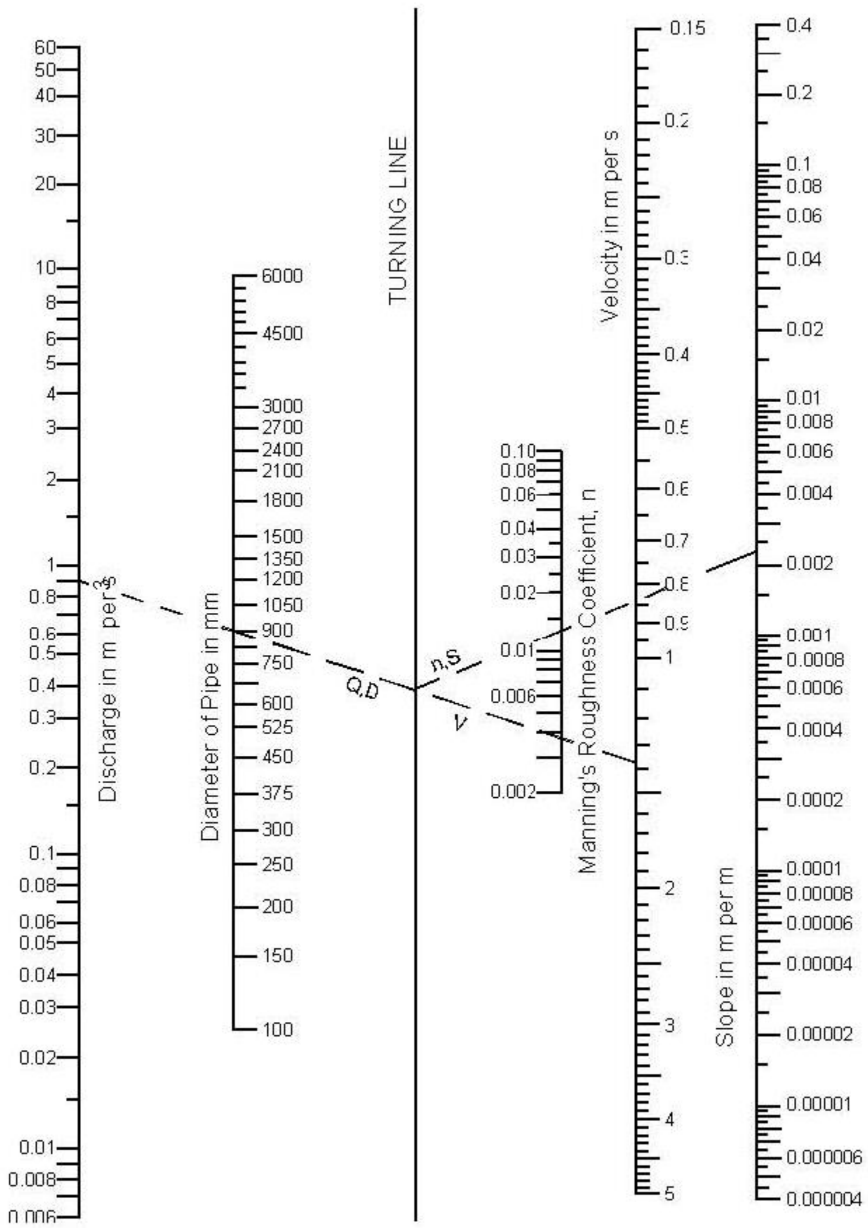
Project:
Designer
Checked:

Enter data in blue cells only yellow cells are calculation cells)

LOCATION				DISCHARGE									
ID	**Pipe Located on:**	**From Station to Station**	**Length of Pipe (m)**	**Source of Drain**	**Catchment Area (km^2)**	**Runoff 'C'**	**C*A (km^2)**	**C*A Laterals (km^2)**	**Total C*A (km^2)**	**Time of Concentration (minutes)**	**Time of Flow in Pipe (minutes)**	**Rainfall Intensity (mm/hr)**	**Runoff (m^3/s)**
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
1	Abuja St	From Ch 2+00 to 3+00	100	A5	0.03	0.25	0.0075	0.0015	0.009	12	1.1	40	0.10
2	Abuja St	From Ch 3+00 to 4+50	150	A6	0.06	0.35	0.021	0.0015	0.0225	15	1.4	40	0.25
3	Abuja St	From Ch 4+50 to 5+50	100	A7	0.047	0.4	0.0188	0.0015	0.0203	18	1.1	40	0.23

Figure 5.9: Preliminary Storm Drain Computation

PIPE SIZING											PROFILE		GRADIENT		
Pipe Diameter (m)	Slope (m/m)	Manning 'n'	K_f	Θ (rad)	h/D	Depth (m)	0.7 D	0.9 D	V (m/s)	$V^2/2g$ (m)	Invert Elevation (m)	Top of Structure (m)	Hydraulic Slope	Elevation	Remarks
15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	
0.375	0.005	0.015	0.29	4.3416	0.7823	0.29	0.26	0.34	1.1	0.0593	50.03	50.405	0.005	50.32	1:5 yr
0.525	0.005	0.015	0.296	4.40483	0.7952	0.42	0.37	0.47	1.4	0.0935	49.53	50.055	0.005	49.95	1:5 yr
0.525	0.005	0.015	0.267	4.09992	0.7305	0.38	0.37	0.47	1.3	0.0903	49.03	49.555	0.005	49.41	1:5 yr



Solution of Manning’s Equation for flow in Pipes

Figure 5.10: Nomograph for Solution of Manning Equation

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6: DRAINAGE OF HIGHWAY PAVEMENTS

6.1 General

Increased emphasis on highway safety has made the problem of removing stormwater runoff from the highway pavement more critical in the design of a highway. Good design involves consideration of roadway gutter, median and inlet capacities. Inlet capacity is discussed in Section 7 of this manual.

The Geometric Design Volume of this manual contains the requirements for minimum grades and cross slopes (respectively) required for good drainage design. The design storm frequency is found on Table 4.6 of Section 4.4.3.

6.2 Gutter Design

6.2.1 Gutter capacity

Gutters shall be designed to keep flooding within the limits as follows (for the 1:10 year design flow, as specified in Section 4.4.3):

a. *Roadways with Shoulders*

The width of the shoulder

b. *Roadways with Kerbs (No Shoulders)*

1.5 metres for 2 lane roadway and 1 traffic lane for multiple roadways.

The capacity of the gutter depends on its cross section, grade and roughness. The Manning equation has been modified to reflect the conditions presented in gutter flow. The modified equation is:

$$Q = \frac{d^{8/3} z^{5/3}}{3.175n} \left(\frac{1}{1+\sqrt{1+z^2}}\right)^{2/3} S^{1/2}$$ **(Equation 6-1)**

Where:

Q = Rate of discharge in cubic metres per second.

Z = reciprocal of the cross slope.

n = Manning's coefficient of roughness

S = Longitudinal slope of m/m.

d = Depth of channel at deepest point in metres.

An alternative to the above formula, which can be readily solved using a standard spreadsheet application, is to use software such as the FHWA Hydraulic Toolbox (Appendix 2) that includes a "Curb and Gutter Analysis" component.

6.2.2 Location and Spacing of Inlets

The location and spacing of inlets depends on the following factors:

- Amount of runoff
- Grade profile
- Geometrics of interchanges and intersections
- Width of flow limitations
- Inlet capacity

In general inlets should be placed at all low points in the gutter grade and at intersections to prevent the gutter flow from crossing traffic lanes of the intersecting road. In urban locations inlets are normally placed upgrade from pedestrian crossings to intercept the gutter flow before it reaches the cross walk. Where pavement surfaces are warped, as at cross streets, ramps or in transitions between super elevated and normal sections, gutter flow should be picked up before the cross slope of the pavement begins to change in order to lessen water flowing across the roadway.

In a sag vertical curve three inlets should be placed, one at the low point and one on each side of this point where the grade elevation is approximately 60 mm higher than at the low point. The additional inlets furnish added capacity to allow for flow bypassing the upgrade inlets and provide a safety factor if the sag inlet becomes clogged. These inlets limit the deposition of sediment on the road in the sag. They also reduce flow arriving at the low point and thereby prevent ponding which could flood the road.

Where a kerbed roadway crosses a bridge, the gutter flow would be intercepted and not be permitted to flow onto the bridge.

Runoff from areas adjacent to the roadbed should be intercepted before reaching the pavement. This applies to water that would normally run onto the highway from side streets or from cut slopes and areas alongside the pavement. Street inlets are inefficient means for intercepting water and should not be used to intercept the runoff that could have been intercepted by more efficient structures such as open channels.

On continuous grades inlets should be spaced so as to limit the spread of the water on the pavement to the criteria used.

With the maximum spread fixed and with a given pavement cross slope and longitudinal slope, the flow in the gutter is also fixed and can be calculated. The spacing of inlets is equal to the length of pavement needed to generate the discharge corresponding to the allowable spread on the pavement. The flow bypassing each inlet must be included in the flow arriving at the next inlet.

Based on construction costs in a particular area, the most economical length of kerb opening inlet or width of grade could be accepted. The size of opening might vary with the slopes and would normally be limited to certain standard sizes of opening. Factors other than maximum allowable spacing must generally be considered and the maximum allowable spacing usually becomes only a guide to spacing of inlets between locations fixed by other considerations.

6.2.3 Median Drainage

The primary purpose of the median is to separate opposing lanes of traffic. The adjacent pavement areas are generally un-kerbed on rural roads and part of the adjacent pavement and the shoulders drain into the median. The median, if over 3 metres wide is generally sloped to a centre swale for drainage, even when a kerbed roadway section is used. The swale should be no deeper than needed to carry the runoff because of the hazard of a deep, steep sided ditch. Side slopes should be no steeper than 6:1 to reduce the hazard to a vehicle out of control. At intervals this stormwater is removed from the median by inlets and carried through a storm drain to an outlet channel. Narrow medians are generally crowned for drainage and the runoff from the median flows into the pavement drainage system.

The design discharge, the longitudinal slope, the capacity of the median channel and the allowable velocity in the median channel determine inlet spacing. In some soils seepage into or drainage of the highway sub grade might affect inlet spacing.

6.2.4 References

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7: Inlet Design

7.1 General

The hydraulic capacity of an inlet depends on its geometry and upon the characteristics of the gutter or channel flow. The following discussion is directed primarily to inlets located in roadway gutters, but the information also applies to other inlet locations.

7.2 Types of Inlets

Gutter inlets can be divided into three major elements, each with many variations. These classes are (a) kerb opening inlets, (b) grate inlets, and (c) combination inlets. Each type of inlet may be installed with or without a depression of the gutter and may be a single or a multiple inlet (two or more closely space inlets acting as a unit). Two identical units placed end to end are called double inlets.

A brief description of the inlet types follows:

a. *Kerb Opening Inlets*

These consist of a vertical opening in the kerb through which the gutter flow passes.

b. *Grate Inlets*

These inlets consist of an opening in the gutter covered by one or more grates.

c. *Combination Inlets*

These units consist of both kerb opening and grate inlet acting as a unit.

7.3 Factors in Inlet Capacity

An inlet located in a sag will eventually take all water reaching it unless it clogs completely or the kerb is over topped. The discussion in this section is restricted to inlets on a grade.

The term inlet capacity is used to mean the catch of the inlet under a given set of conditions rather than the maximum water that can be intercepted by the inlet if the discharge is increased without limit. The efficiency of an inlet is the discharge intercepted by the inlet (Q_i) divided by the flow in the gutter (Q). The discharge that bypasses the inlet (Q_b) is termed carry over.

A major factor in the capacity of a kerb opening inlet is the depth of water in the gutter immediately adjacent to the entrance. The capacity of an efficient grate inlet depends principally upon the quantity of water flowing in the section formed by increasing the grate width upstream. Therefore, an increase in longitudinal slope reduces inlet capacity (except where deflectors are used and for some grates) while an increase in transverse (cross) slope increases inlet capacity. Increase in length of a kerb opening inlet and increases in width of a grate opening increases the capacity of the inlet. For grate inlets the efficiency of the grate opening is an important factor in inlet capacity.

For a kerb opening inlet depressing the gutter increases the capacity of the inlet. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet.

7.3.1 Grate Inlets

Grate inlets are recommended where clogging is not a problem and the inlets are not in the traffic stream where noise and depressions are objectionable. An efficient grate will intercept all water flowing within the grate width. For long grates on flat slopes or for depressed grates, the intake on the length of the grate may be computed by treating that edge as a kerb inlet. An efficient grate inlet has all rectangular bars parallel to the flow and opening covering at least 50% of the intake width. The required length of opening can be computed using the experimental formula:

$$L_b = \frac{V}{2}(d + d_b)^{1/2}$$ **(Equation 7-1)**

Where:

L_b = Length of clear opening in metres

V = mean approach velocity, in width of grate in metres per second

d = Depth of flow at the kerb in metres

d_b = Depth of bar, in metres

A grate inlet in a sag vertical curve may act as a weir or an orifice depending upon the depth of water above the grate.

For a depth of water up to about 120 mm above the top of the grate an inlet will operate as a weir. The crest length will be equal to its outside perimeter (P) along which the flow enters. The discharge intercepted by the grate is given by:

$$Q = 0.15Pd^{1.5}$$ **(Equation 7-2)**

Where:

Q = Discharge intercepted in cubic metres

P = Perimeter of grate neglecting bars and side against the kerb in metres

d = Depth of water at grate in metres

When the depth of water exceeds 430 mm the grate tends to operate as an orifice and the interpreted flow is given by:

$$Q = 0.15Ad^{0.5}$$ **(Equation 7-3)**

Where:

Q = Discharge intercepted in cubic metres

A = Clear opening of grate in square metres

d = depth of water in metres

Because turbulence over the grate and tendency of trash to collect on the grate, the clear opening of the grate should be reduced by 75% when computing the intercepted discharge and when design considerations warrant the clear area should be reduced by 50%.

Figure 7.1 is a graphic solution to the above expressions.

7.3.2 Kerb Opening Inlets

Kerb opening inlets are used in many locations because they offer little interference to traffic and are relatively free from clogging by debris.

The best type of kerb opening inlet has a cantilevered top slab without supports in the opening and a depression of the gutter flow line of at least 50 mm. The length of the opening can be varied with the amount of water to be intercepted.

The capacity of a kerb opening inlet depends upon the length of opening the depression of the inlet lip, the depth of flow at the kerb line in the gutter and both the cross slope and longitudinal slope of the gutter. Figure 7.2 can be used to determine the hydraulic capacities of depressed kerb opening inlets.

When the inlet is not depressed and is not located in a sag vertical curve, the approximate capacity can be computed from the weir equation:

$$Q = 0.15Ld^{1.5}$$ **(Equation 7-4)**

Where:

Q = Discharge intercepted in cubic metres

d = depth of water above inlet lip in metres

L = Length of clear opening in metres

When the depth at the opening exceeds 1.4 times the opening height, the capacity may be computed by the equation:

$$Q = 0.0057A\left[2g(d - \frac{h}{2})\right] \quad \textbf{(Equation 7-5)}$$

Where:

Q = Discharge intercepted in cubic metres

A = Area of inlet opening in square metres

h = Height of opening in metres

d = depth of water above inlet lip in metres

g = 9.81 m per second^2

6. Combination Inlets

The capacity of an unclogged combination inlet on a continuous grade, using an efficient grate is not appreciably greater than that of the grate alone.

The capacity is computed by ignoring the kerb opening and computing the capacity of the grate opening alone.

Where an inefficient grate is used on a continuous grade, placing the kerb opening upstream from the grate can often increase the capacity of a combination inlet. This has the added advantages of intercepting some debris that might otherwise clog the grate thereby avoiding the situation where debris caught on the grate deflects water away from the kerb opening. An unclogged efficient grate is efficient in any position relative to the kerb opening.

In a sag, combination inlets are very desirable. The kerb opening provides a relief opening if the grate should become clogged. To compute the capacity consider the grate only, but do not divide the perimeter area by 2 when computing the capacity.

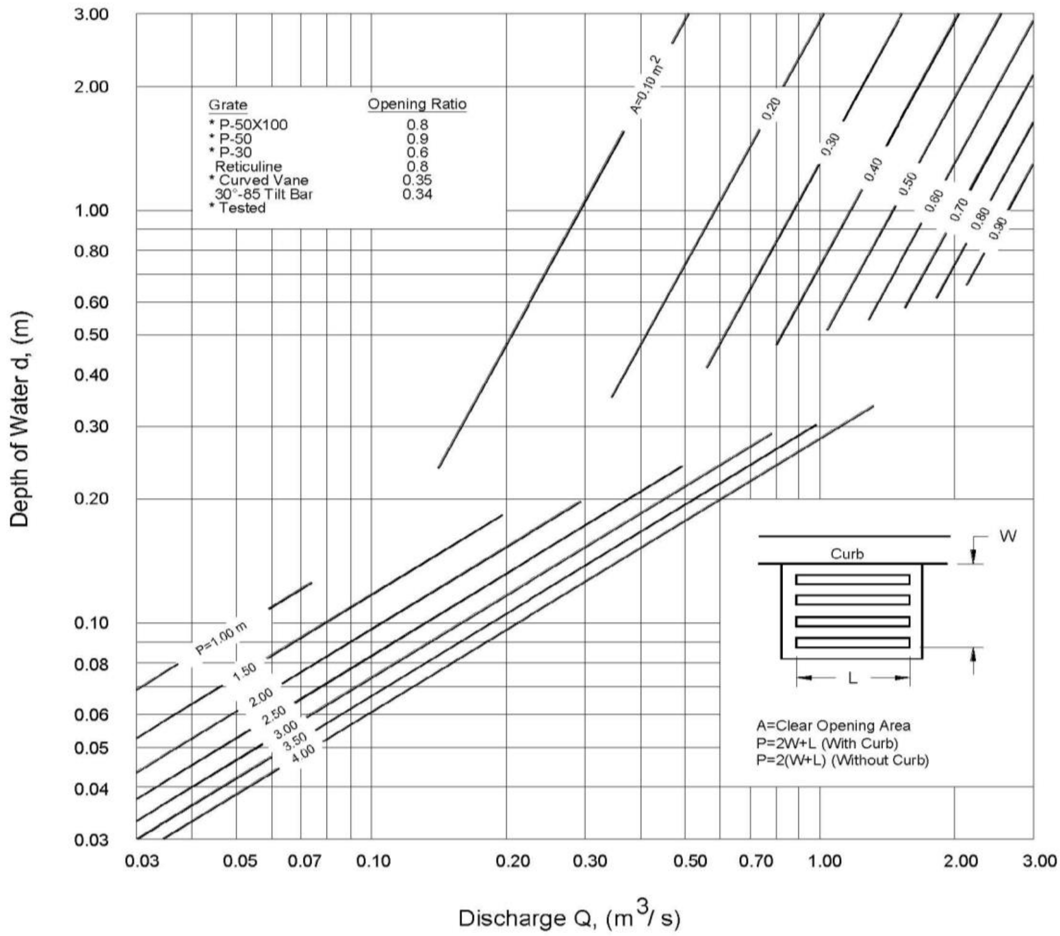

Figure 7.1: Hydraulic Capacity of Grate Inlet in Sump

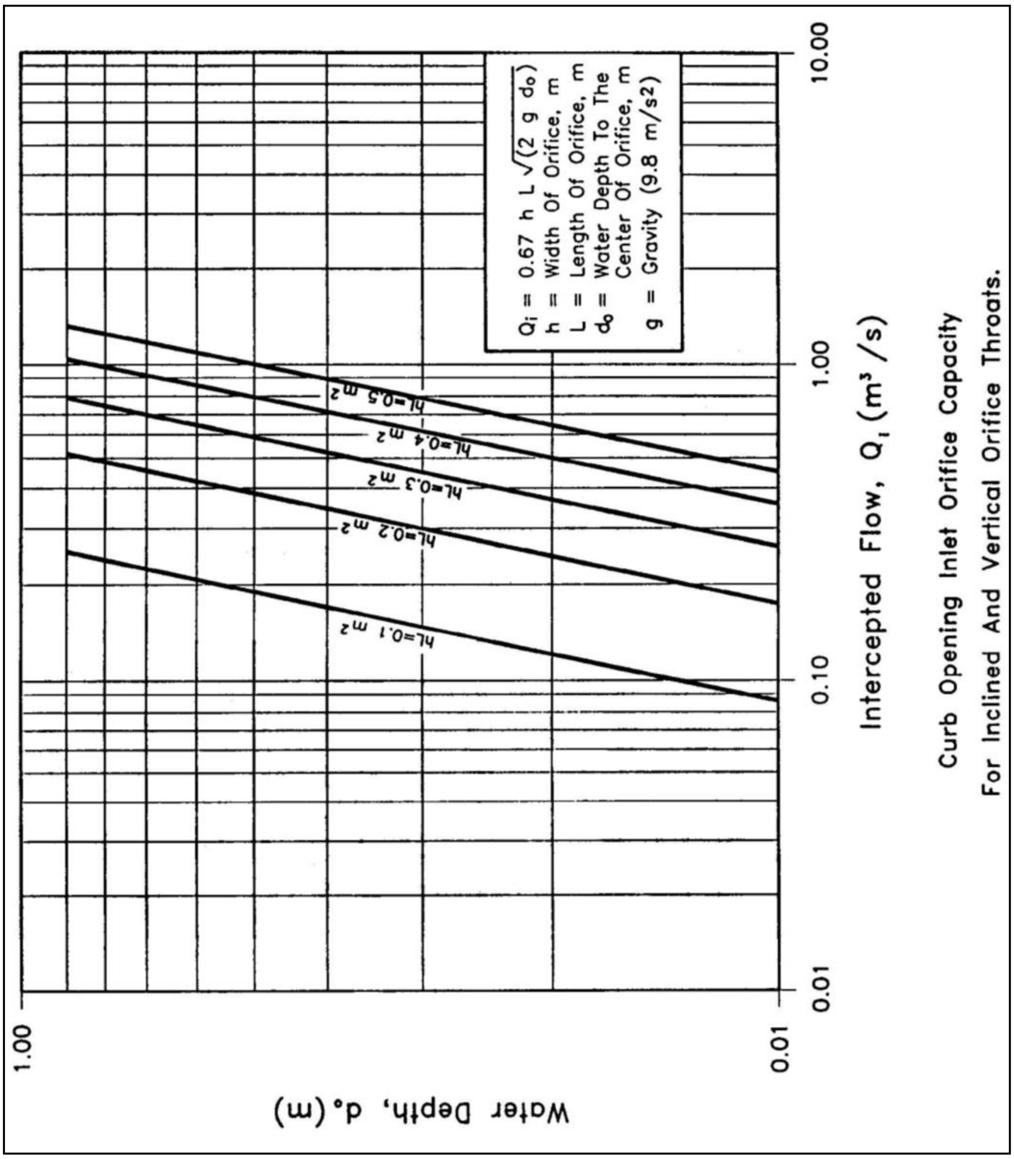

Figure 7.2: Capacities of Kerb Inlets

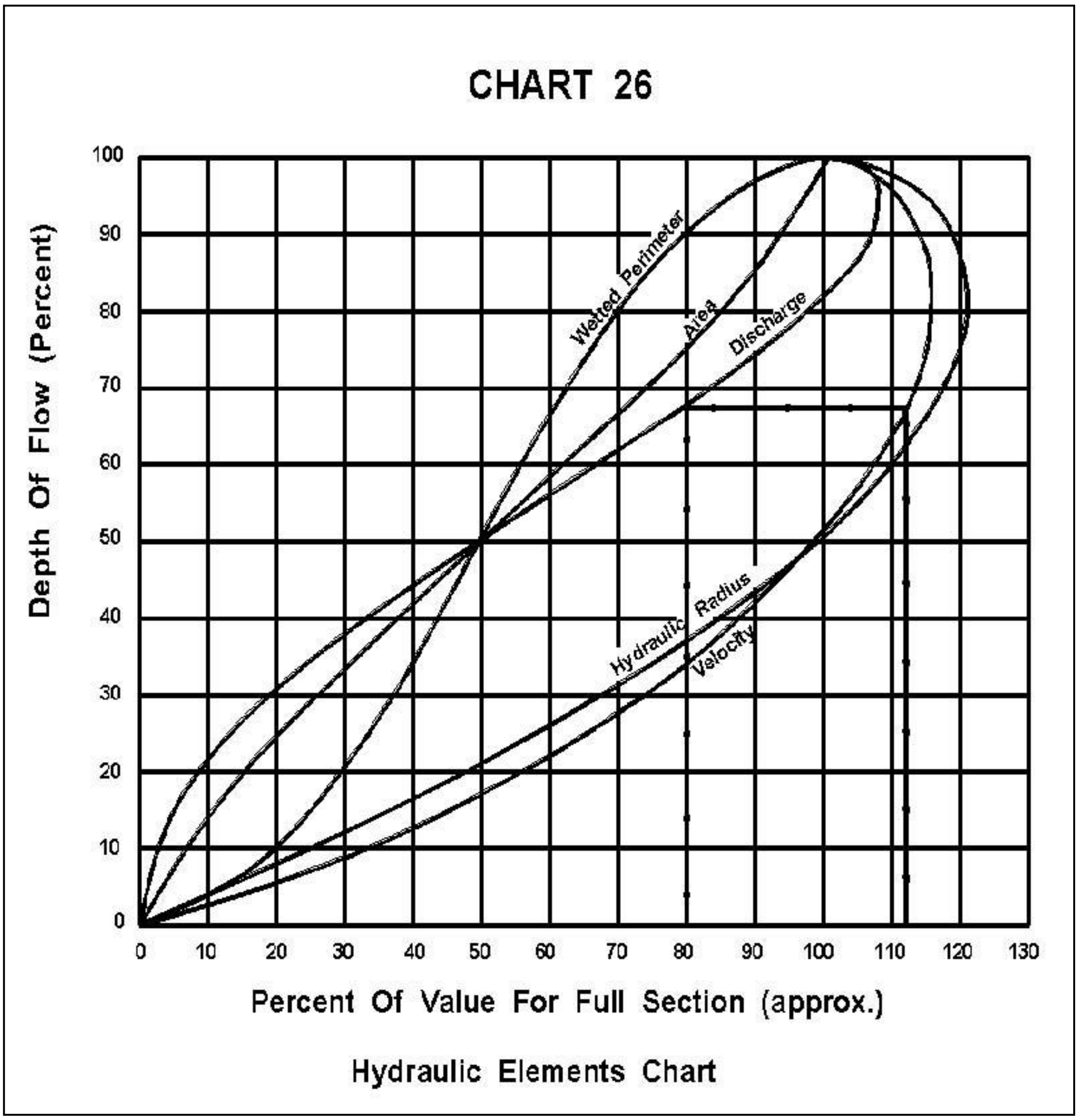

Figure 7.3: Ratio Depth of Flow to Diameter of Pipe

7.3.3 References

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8: SUBSURFACE DRAINAGE

8.1 General

The aim of subsurface drainage is the removal of detrimental amounts of ground water to provide a stable roadbed and side slopes.

The solution of subsurface drainage problems often calls for knowledge of geology and the application of soil mechanics.

There are many variables and uncertainties as to the actual subsurface conditions. In general, the more obvious subsurface drainage problems can be anticipated in design, the less obvious are frequently discovered during construction.

Extensive exploration may be required to obtain the design variables with reasonable accuracy. For these reasons designs are based on local experience and empirical rules, which have given satisfactory results.

8.2 Subsurface Discharge

Ground water, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. The rates at which ground water can be removed, or the subsurface discharge, depends on the effective hydraulic bed and on the permeability, depth, slope, thickness and extend of the water bearing formation (the aquifer). The discharge can be obtained by analytical methods. Such methods, however, are usually cumbersome and unsatisfactory: field explorations will yield better results.

8.2.1 Preliminary Investigations

Field investigations may include: (a) soils and geological studies, (b) borings, pits or trenches to find the elevation, depth and extent of the aquifer, (c) inspection of cut slopes in the immediate vicinity, and (d) measurements of the quantity of discharge, when feasible.

Preliminary investigations must be as thorough as possible, recognizing that further information is sometimes uncovered during construction. Where an existing road is part of the new construction, the presence and origin of ground water is often known or easily detected. Explorations, therefore, are likely to be lesser in scope and cost than explorations for a project on a new alignment. In slope stability questions, and other problems of equal importance, extensive knowledge of subsurface conditions is required.

In general, explorations should be made during the wet season. An exception would be where seepage from irrigation sources occurs.

Ground water difficulties frequently stem from perceived water. Pumped water supply wells often give unreliable indications of the water table and such data should be used with caution.

8.2.2 Types of Subsurface Drains

Three types of subsurface drains are used: standard pipe under drains, bored horizontal drains, and stabilization trenches. French drains have been found unreliable and are not recommended.

a. Pipe Under Drain

The standard pipes under drains consist of a perforated pipe at the bottom of a narrow trench backfilled with filter material. This type is commonly used as an under drain in these cases:

(i) Along the toe of a cut slope to intercept seepage when slope stability is not in question.

(ii) Along the toe of a fill on the side from which ground water originates.

(iii) across the roadway at the downhill end of a cut.

In fill foundation areas, a system of under drains in a herringbone or other effective pattern may be required depending on the quantity of water, type of material and area to be stabilized.

When it is not feasible to place under drains at a sufficient depth to keep the water from the roadbed, a pervious blanket may be used over the basement soil along with under drains.

Two methods of installing pipe under drains are used: one with the filter material carried to the sub grade level and one with a topping of earth over the filled material. The first alternative shall be used under a paved area and the second under an unpaved area.

b. Bored Horizontal Drains

Horizontal drains are 50 mm perforated metal pipes placed in drill holes bored into the aquifer or water bearing formation.

They are installed in cut slopes and under fills, more to guard against slides rather than to prevent saturation of the roadbed. They may be used in varying lengths up to 120 metres on grades that range from 5 to 20%.

c. Stabilization Trenches

These are wide trenches with sloping sides having a blanket of filter material 900 mm thick on the bottom and side slopes of the trench.

One or more perforated drain pipe 200 to 300 mm in diameter are set at the bottom of the trench.

Stabilization trenches are placed in swales, ravines and under side hill fills to stabilize waterlogged fill foundation areas that are well defined. In swales and ravines they are approximately normal to the highway. On side hills they are parallel to the centreline, more or less. Sometimes conditions dictate construction on a skew or with a tee type or herringbone configuration.

Trenches should be wide enough to permit the use of earth moving equipment. The side slopes commonly used are 2:1.

8.2.3 Design Requirements for Subsurface Drains (Flow Drains/ Fin Drains)

a. Size and Length Requirements

The minimum inside pipe diameter for a standard pipe under drain shall be 150 mm for lengths of 150 metres or less.

As a general rule this size is adequate as a collector or lateral in most soils. For lengths exceeding 150 metres the minimum diameter shall be 200 mm.

b. Separation of Drainage

Surface drainage shall not be permitted to discharge into the under drain. The discharge form an under drain into a storm drain or a culvert however is permissible if the out fill for the under drain is not under the hydraulic gradient for the storm sewer system.

c. Clean Cuts

A terminal clean cuts should be installed at the upper end of the under drain. This is made by bringing the pipe to ground level on a 45 degree angle.

Intermediate inspection wells are required at 150 metre intervals. They may consist of a vertical riser with a light cast iron cover brought to ground level. The diameter of the riser shall be at least the diameter of the conduit.

d. Grade Requirements

In general the grade should not be flatter than 0.5%. If this slope is unobtainable grades of 0.2% for laterals and 0.25% for mains will be acceptable.

e. Depth and Spacing of Under Drains

The depth of the under drain depends on the permeability of the soil, the elevation of the water table, and the amount of drawdown needed to ensure stability. Whenever practicable, and under drain pipe should be set in the impervious zone below the aquifer. Table 8.1 gives suggested depth and spacing of under drains according to soil type. It is only a guide and should not be considered a substitute for filed observations or local experience.

Table 8.1: Suggested Spacing of Under Drains for Various Soil Types and Drain Depths

Soil Classes	**Soil composition**			**Drain Spacing (Metres)**			
	Percent Sand	**Percent Silt**	**Percent Clay**	**0.90 m Deep**	**1.2 m Deep**	**1.5 m Deep**	**1.8 m Deep**
Clean Sand	80 – 100	0 - 20	0 – 20	33 - 45	45 – 60		
Sandy Loam	50 – 80	0 – 50	0 – 20	15 – 30	30 – 45		
Loam	30 – 50	30 – 50	0 – 20	9 – 18	12 – 24	15 - 30	18 – 36
Clay Loam	20 – 50	20 – 50	20 – 30	6 – 12	7.5 – 15	9 – 18	12 – 24
Sandy Clay	50 – 70	0 – 20	30 – 50	4.5 – 9	6 – 12	7.5 – 15	9 – 18
Silt Clay	0 – 20	50 – 70	30 – 50	3 – 7.5	4.5 – 9	6 – 12	7.5 – 15
Silt	0 - 50	0 – 50	30 - 100	4.5 (max)	6 (max)	7.5 (max)	12 (max)

f. Outlets

Outlets should be provided at intervals of not more than 300 metres.

8.2.4 Types of Conduits

The aim of any under drain installation is long-term effectiveness. The aim is associated with filtering ability, durability, strength of conduit and cost, mainly in that order. In choosing between pipes of different types the key considerations are filtering ability and durability. Pipe cost assumes secondary importance because it is a minor part of the under drain investment.

Because open joint pipes tend to admit excessive solids, pipes with perforated walls and closed joints are recommended. They may be made of concrete, uPVC, clay or fibre.

8.2.5 References

U.S. Department of Transportation, (1989) "Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12.

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The South African Roads Agency Limited, 2013, Drainage Manual 6th Edition.

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VicRoads (Australia), 2003, Road Design Guidelines, Part 7 – Drainage.

Austroads, September 2008, (Austroads Publication No. AGRD05/08), Guide to Road Design, Part 5: Drainage Design.

UK Highways Agency, 1996-2009, Design Manual for Roads and Bridges (Various Advice Notes).

9: Appendix 1A: Intensity-Duration-Frequency Charts & Climate Information

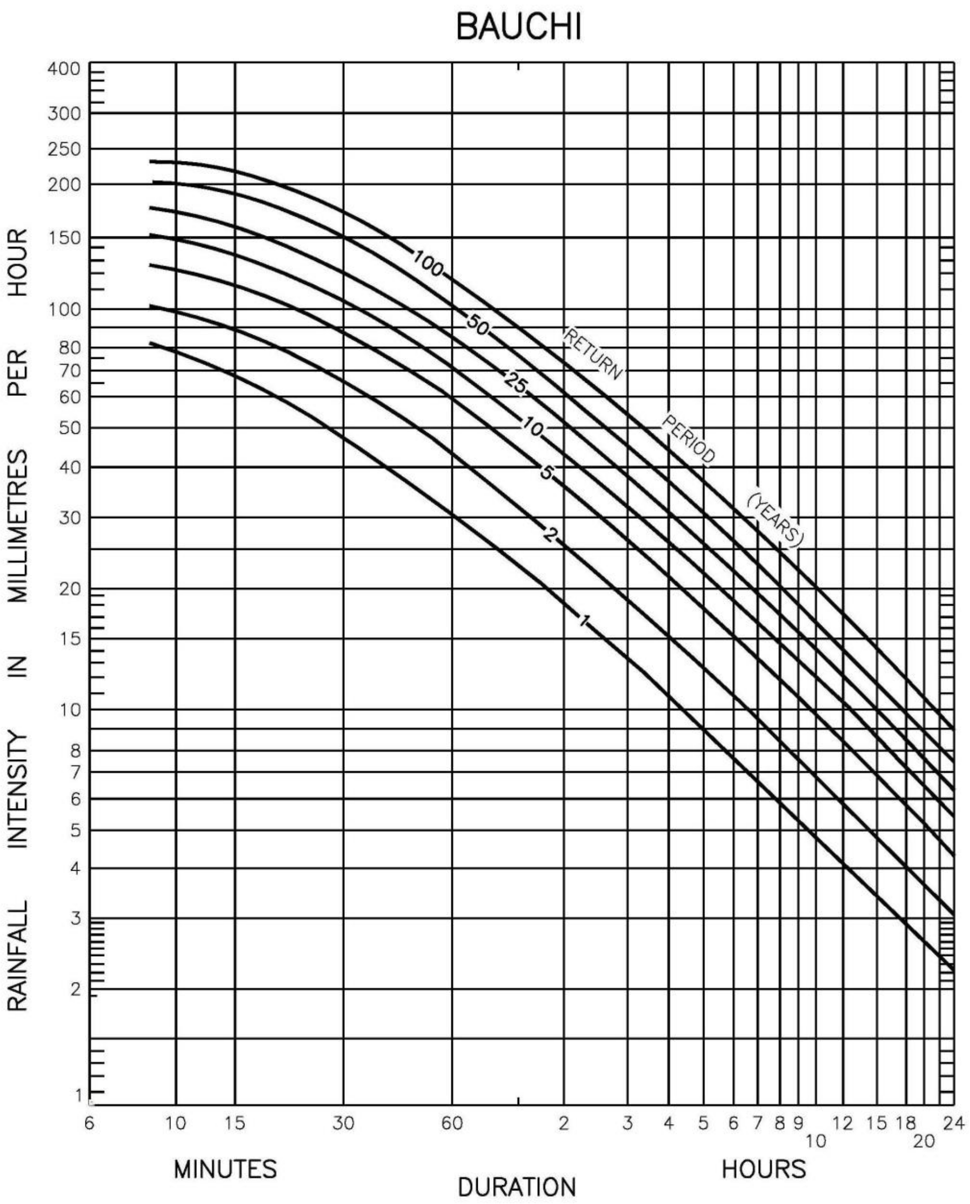


Figure 9.1: Intensity-Duration-Frequency Curves for Bauchi

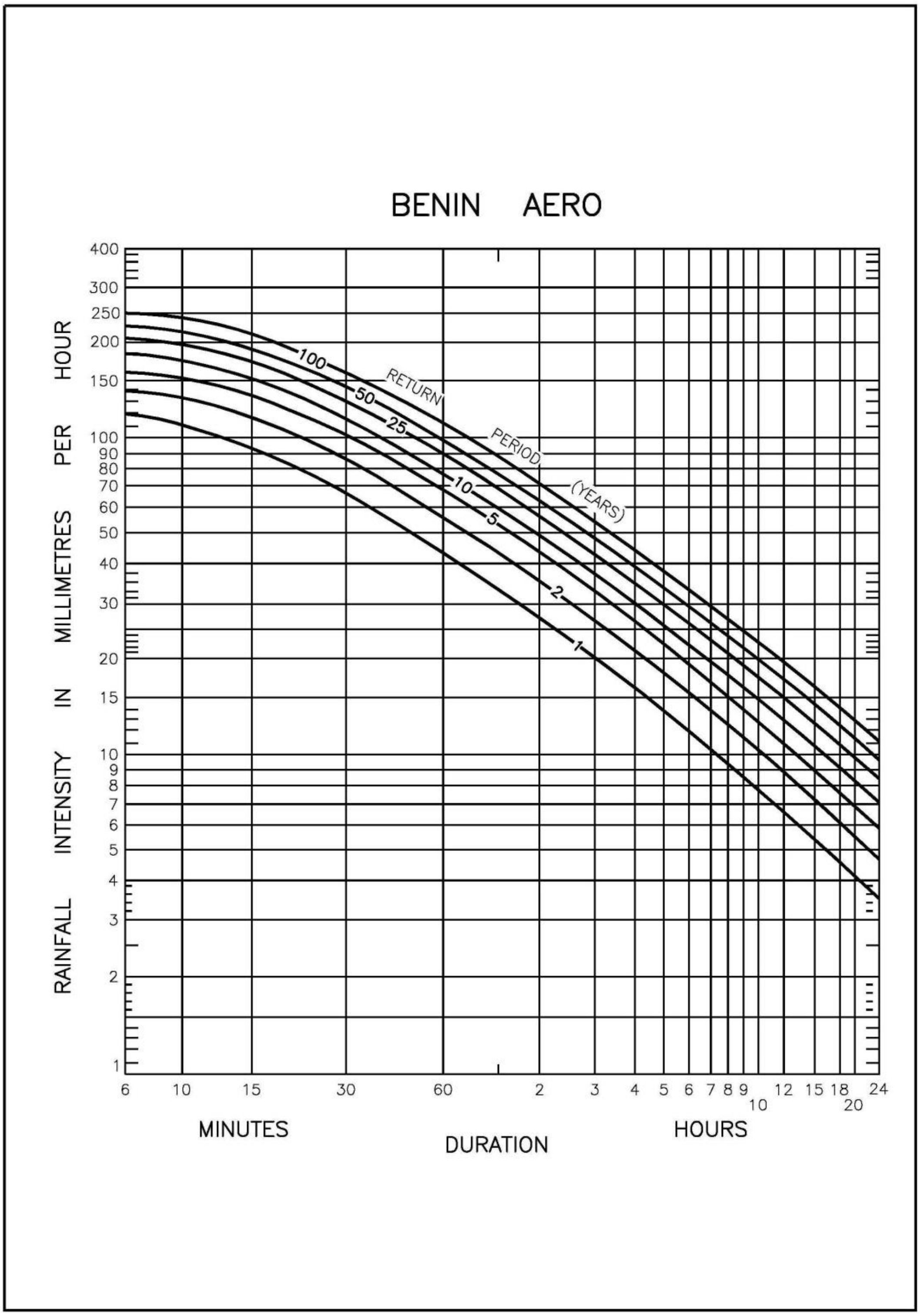


Figure 9.2: Intensity-Duration-Frequency Curves for Benin Aero

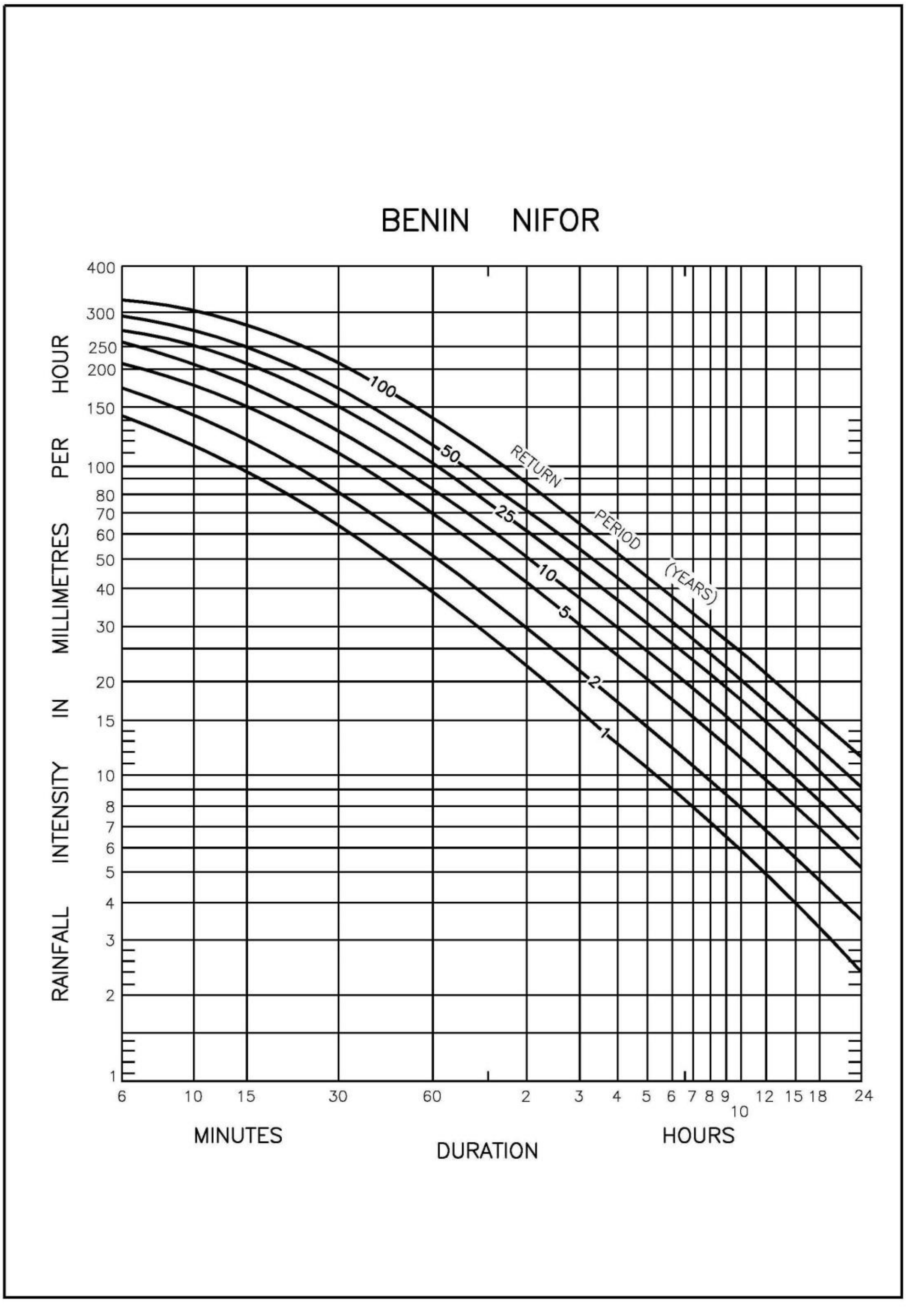


Figure 9.3: Intensity-Duration-Frequency Curves for Benin Nifor

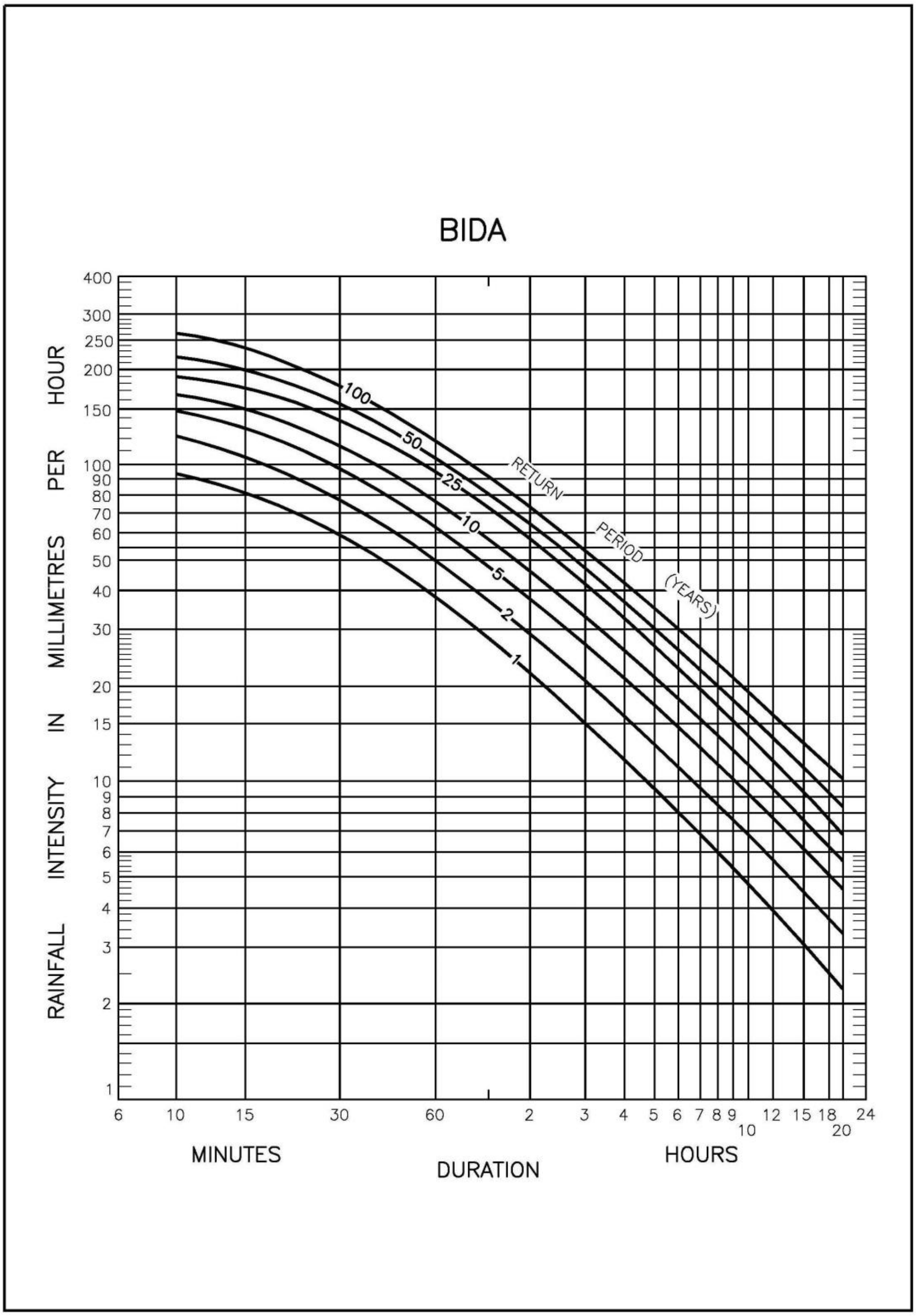


Figure 9.4: Intensity-Duration-Frequency Curves for Bida

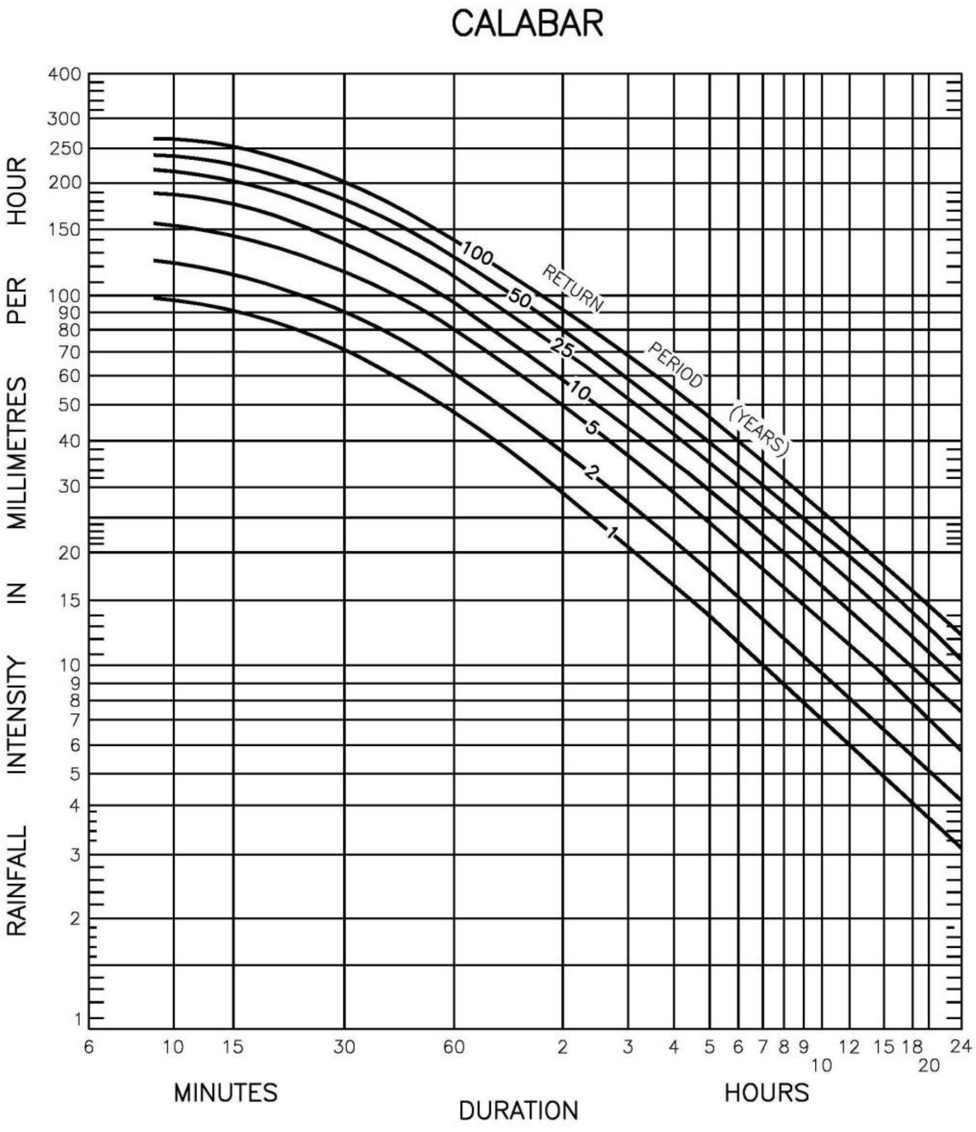


Figure 9.5: Intensity-Duration-Frequency Curves for Calabar

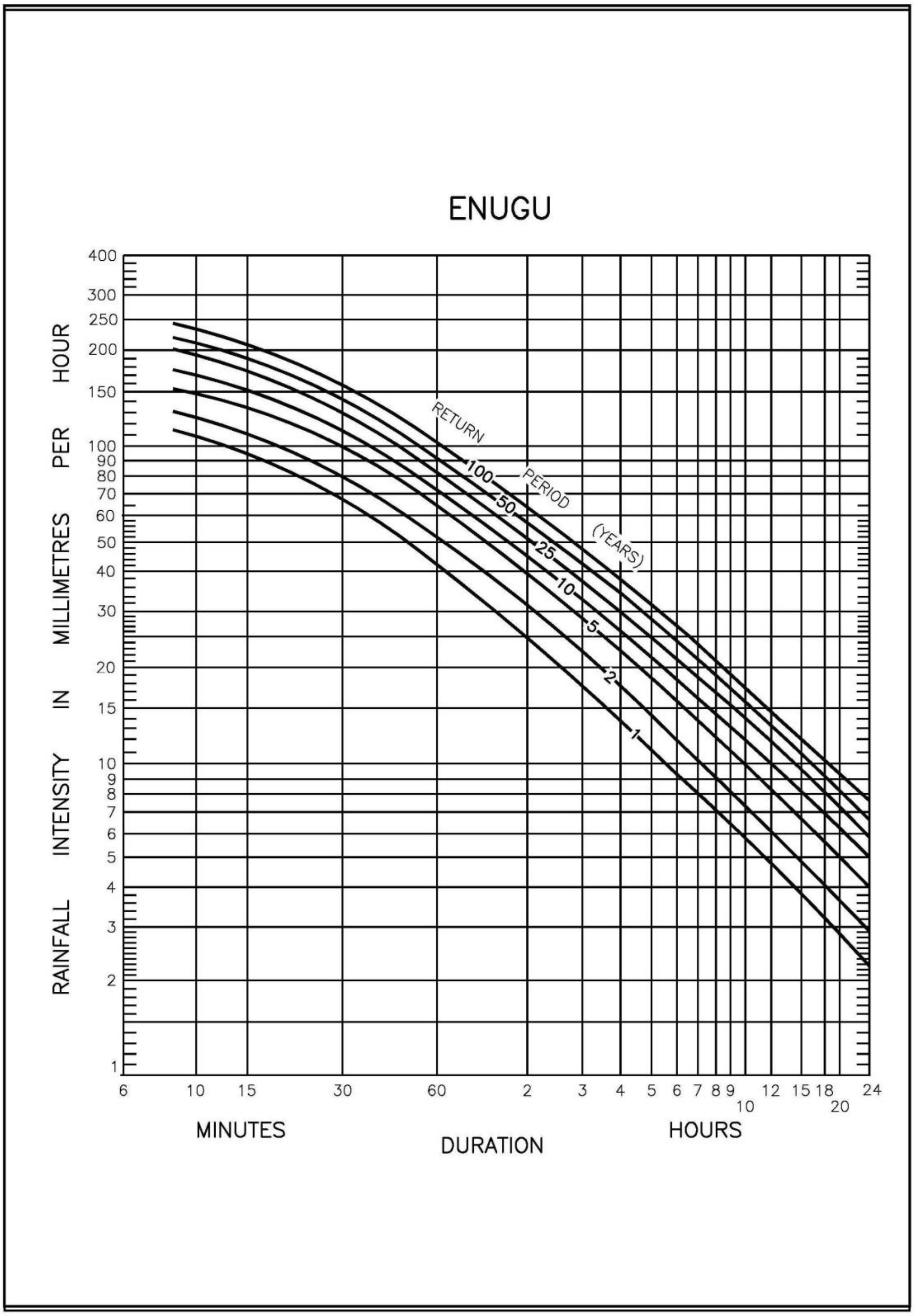


Figure 9.6: Intensity-Duration-Frequency Curves for Enugu

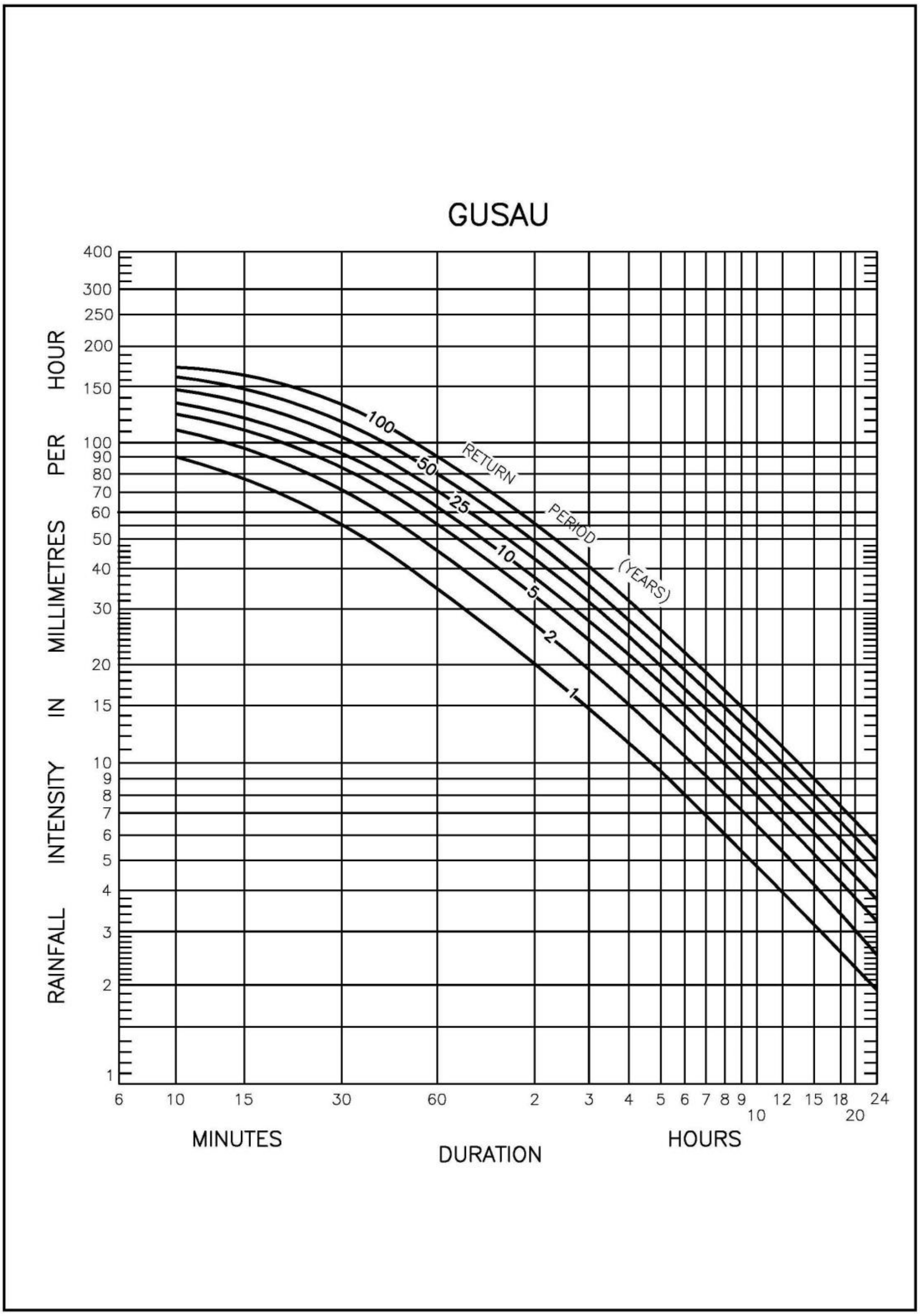


Figure 9.7: Intensity-Duration-Frequency Curves for Gusau

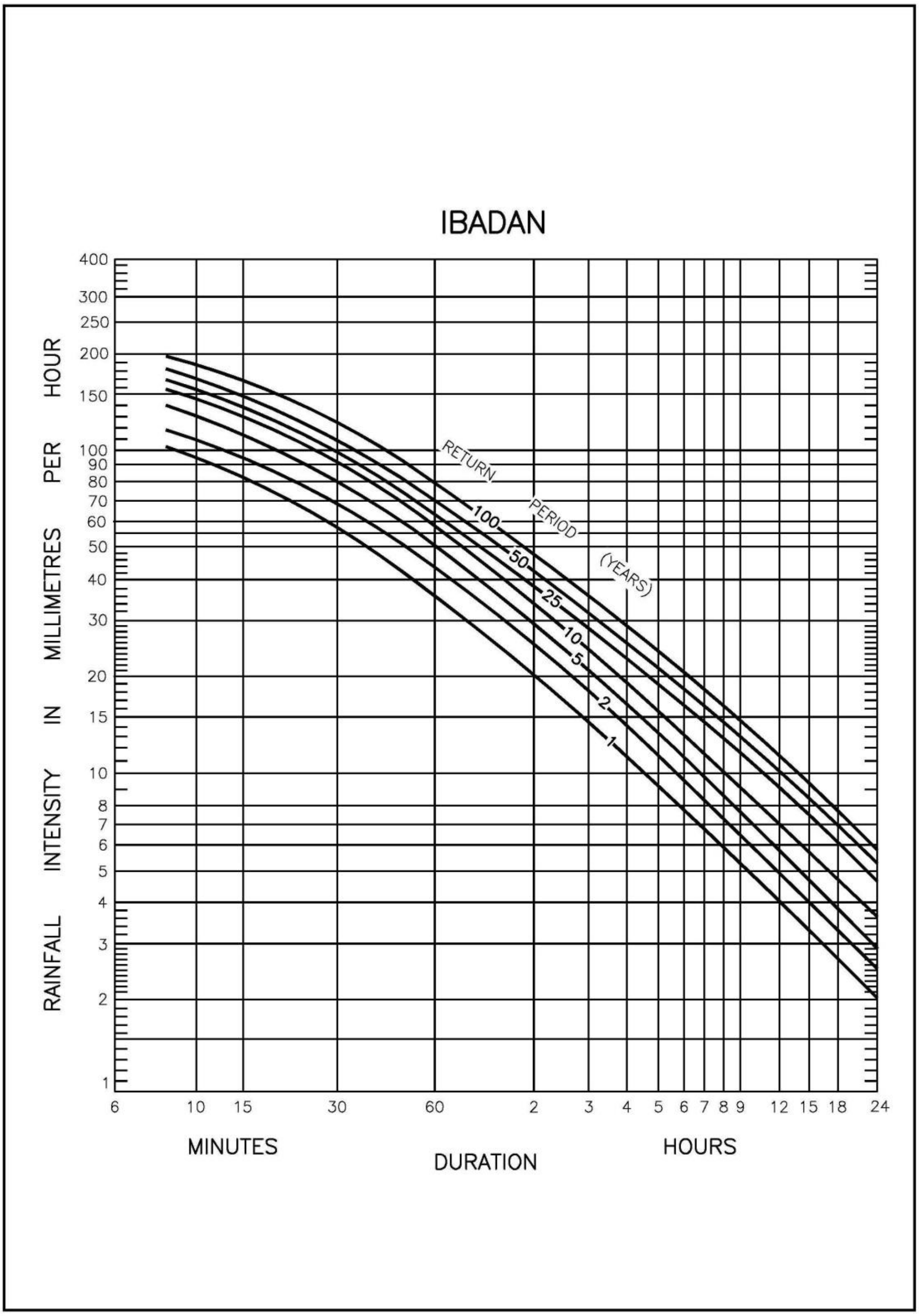


Figure 9.8: Intensity-Duration-Frequency Curves for Ibadan

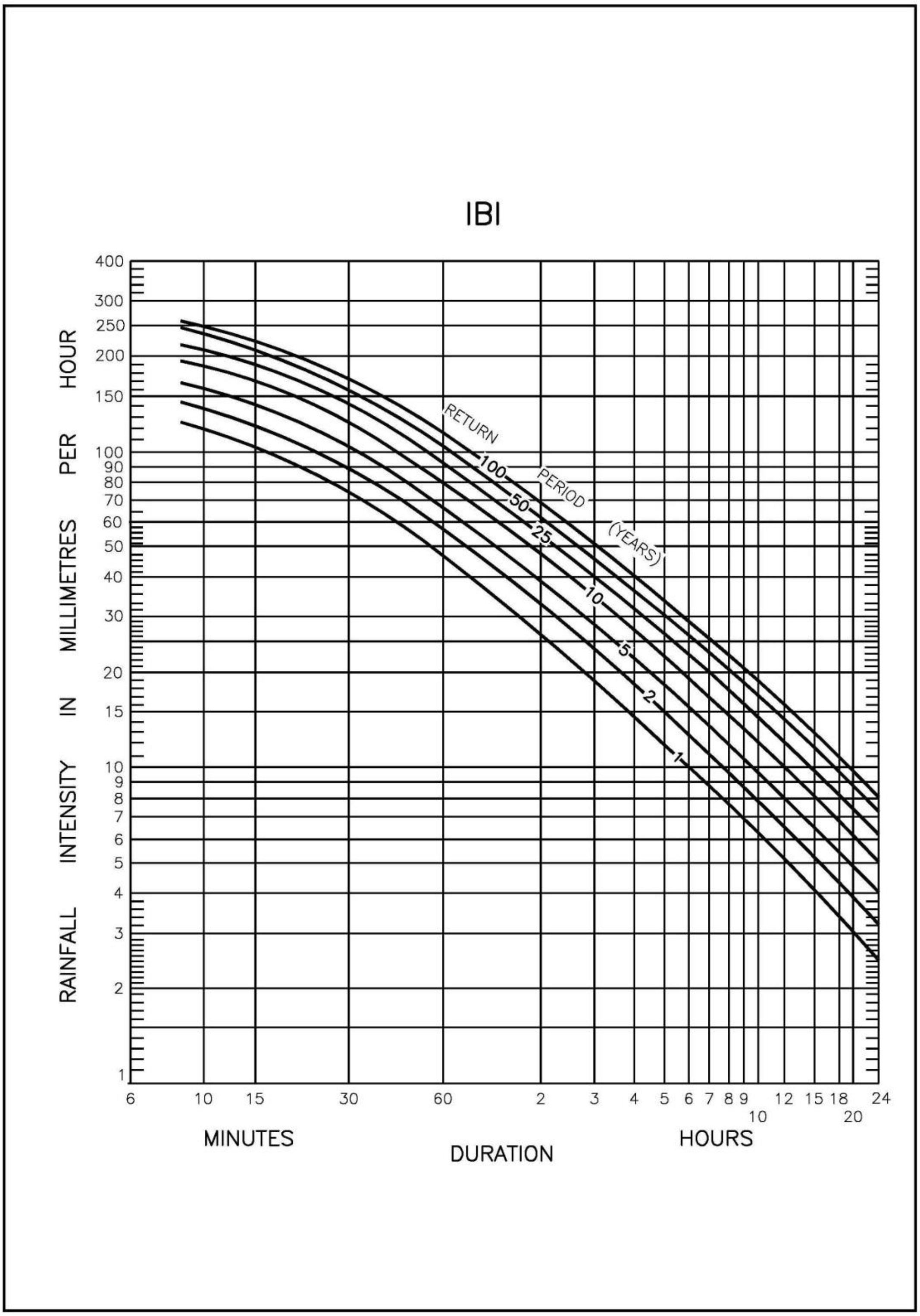


Figure 9.9: Intensity-Duration-Frequency Curves for Ibi

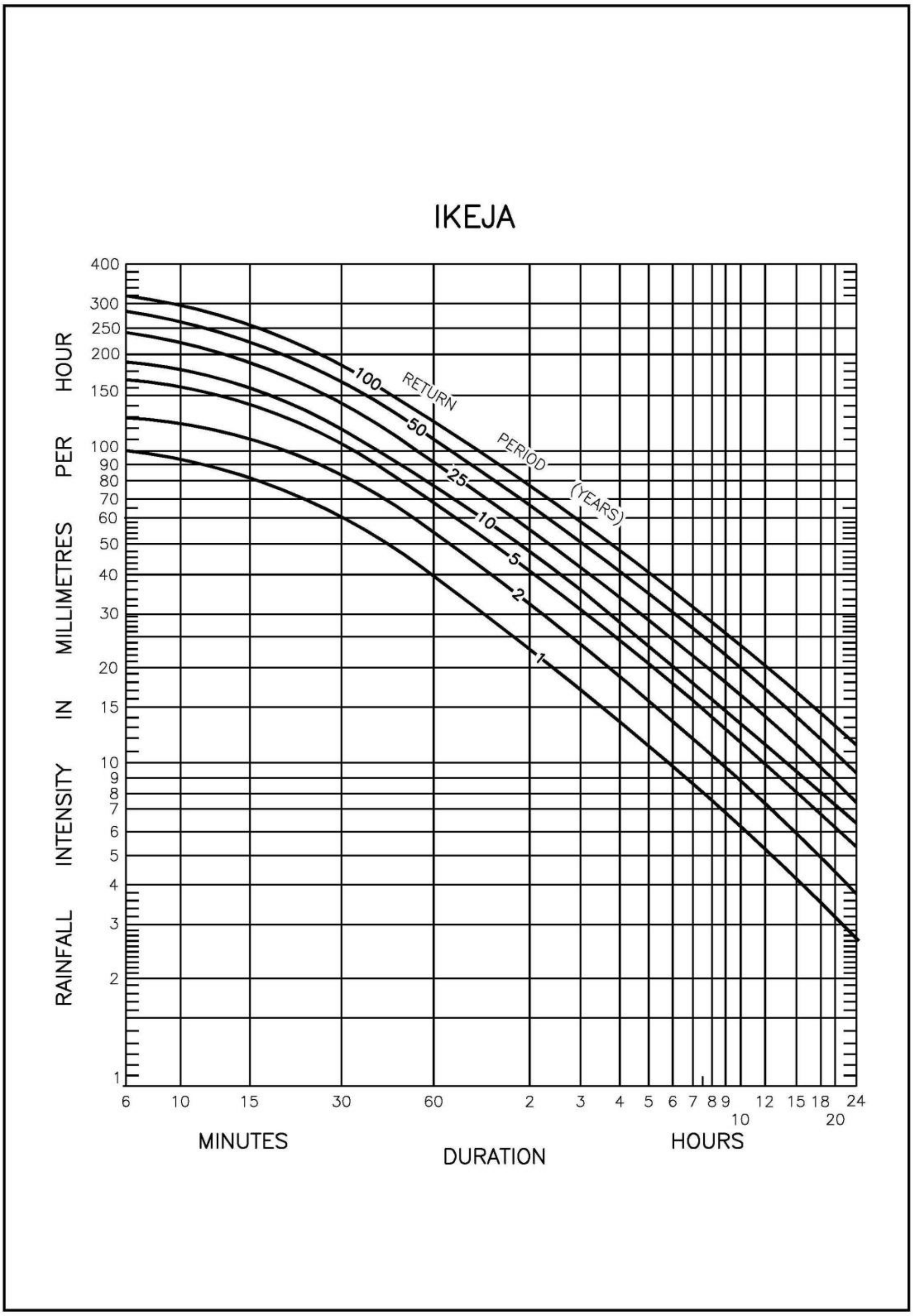


Figure 9.10: Intensity-Duration-Frequency Curves for Ikeja

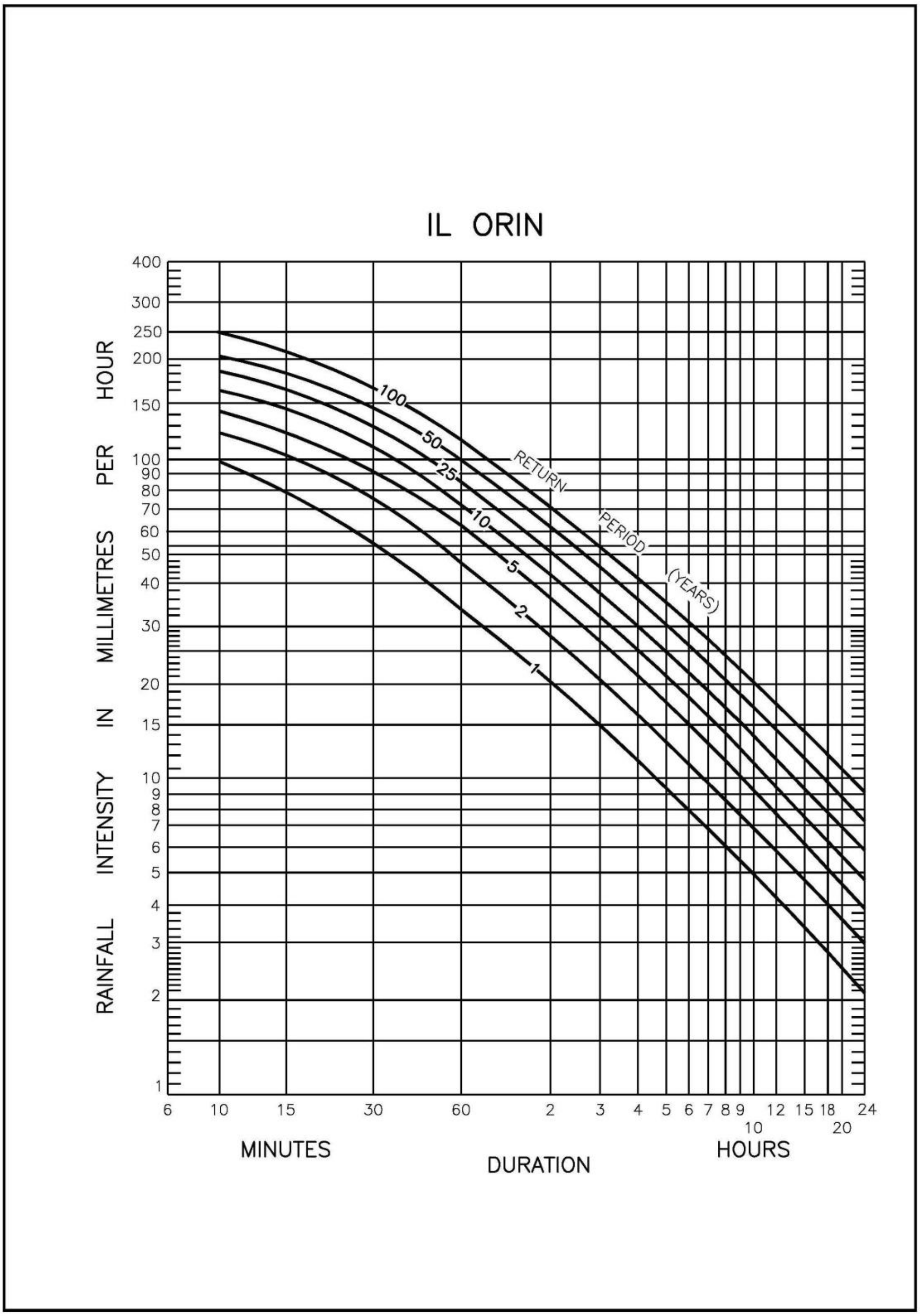


Figure 9.11: Intensity-Duration-Frequency Curves for Ilorin

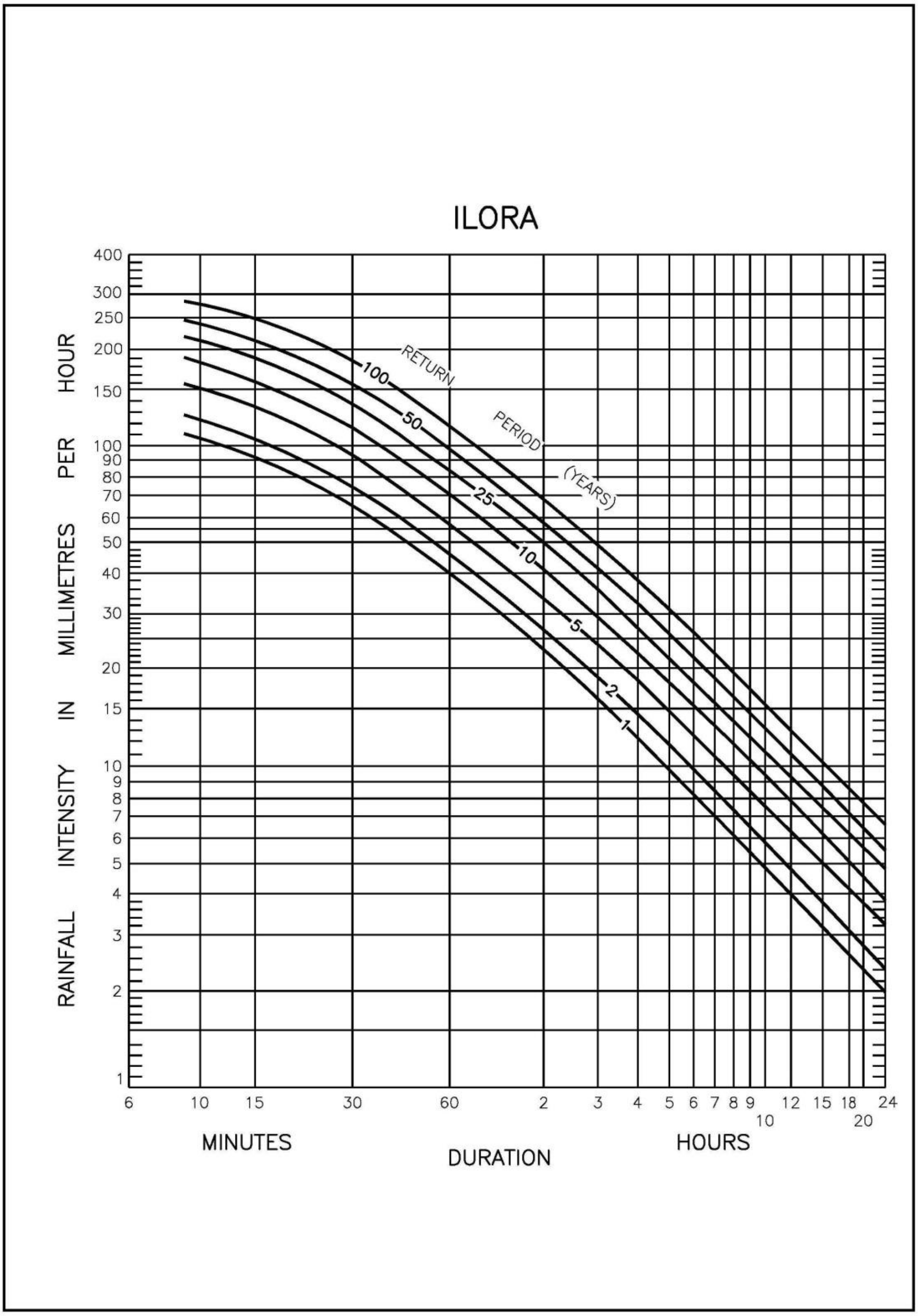


Figure 9.12: Intensity-Duration-Frequency Curves for Ilora

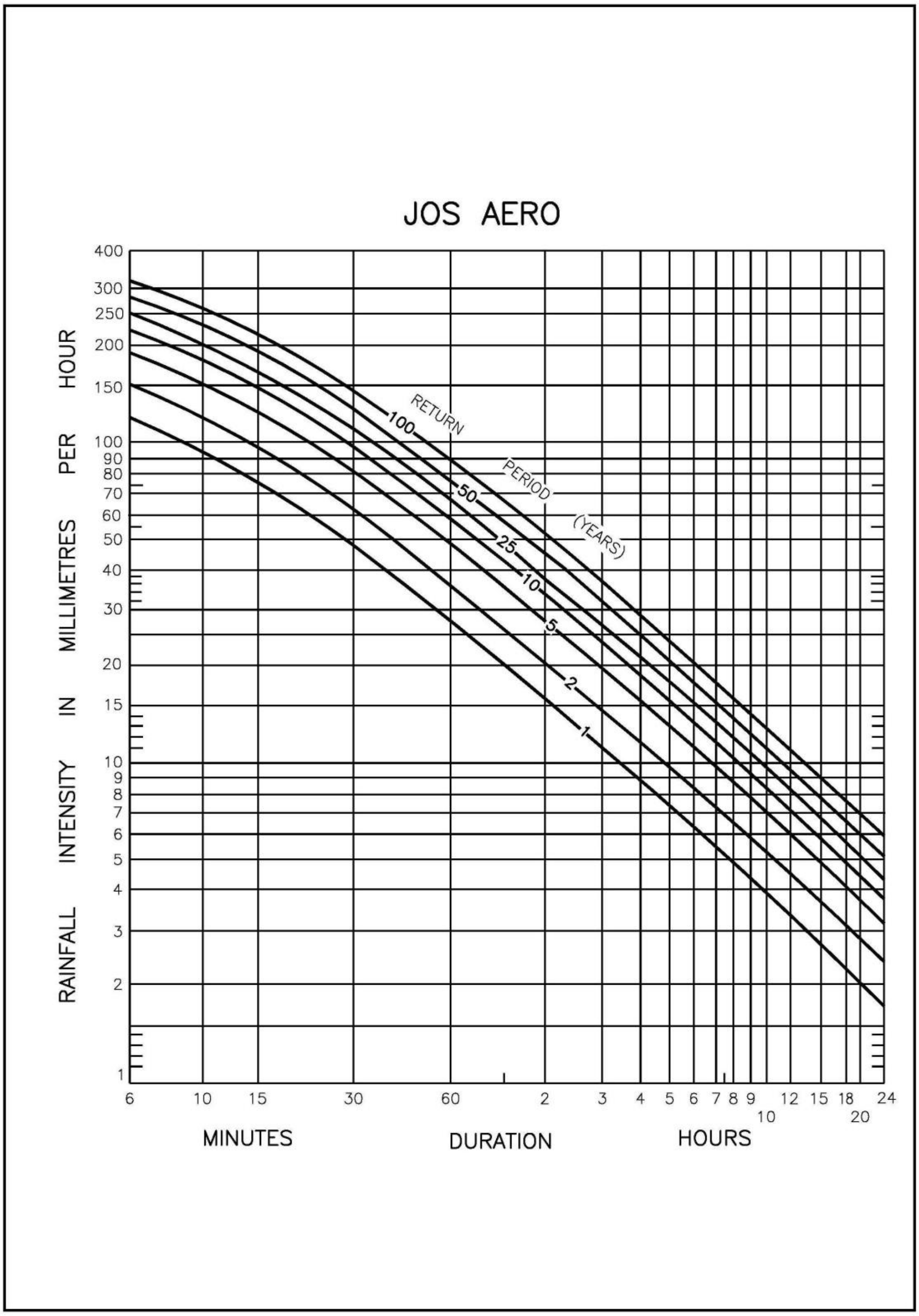


Figure 9.13: Intensity-Duration-Frequency Curves for Jos Aero

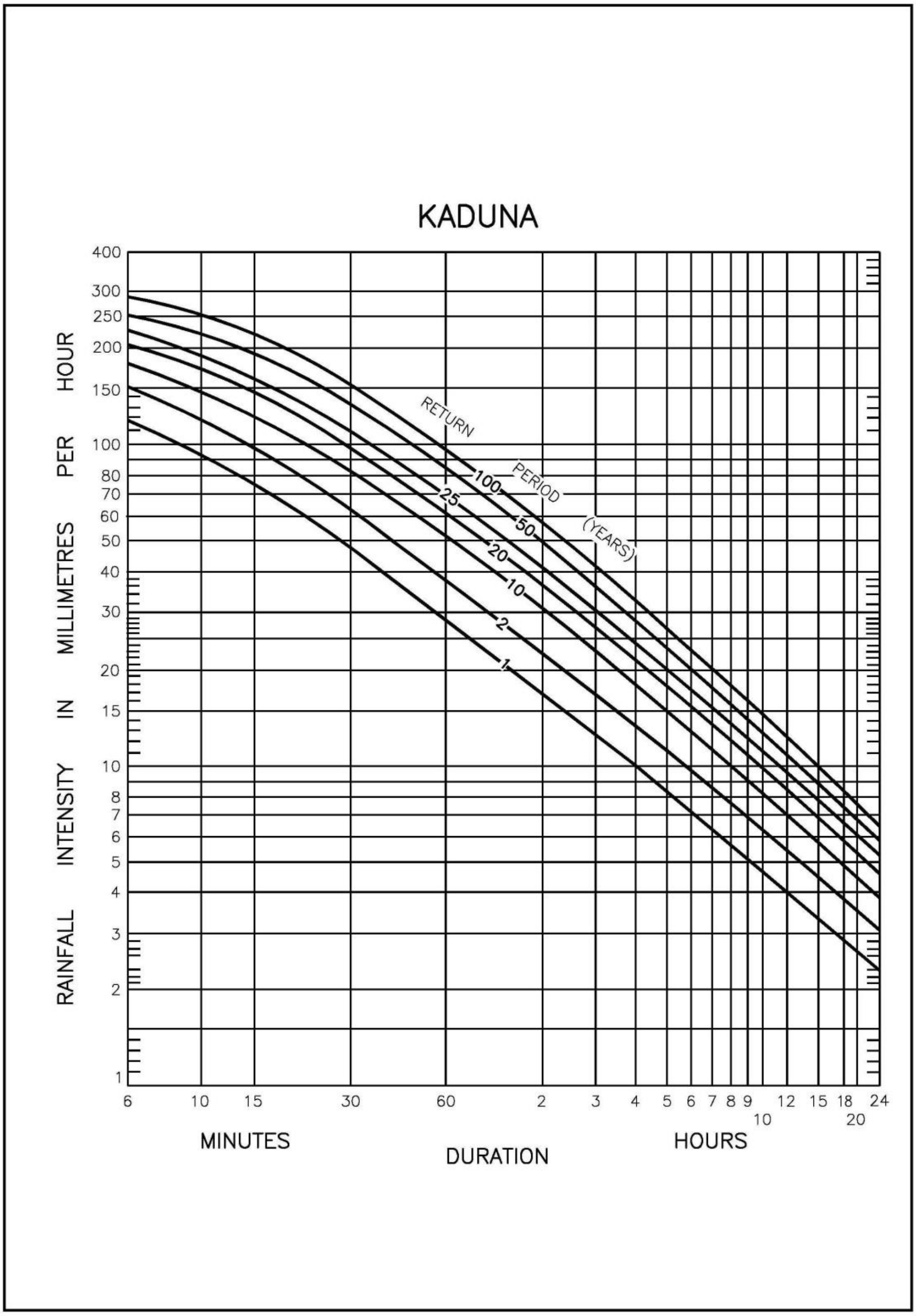


Figure 9.14: Intensity-Duration-Frequency Curves for Kaduna

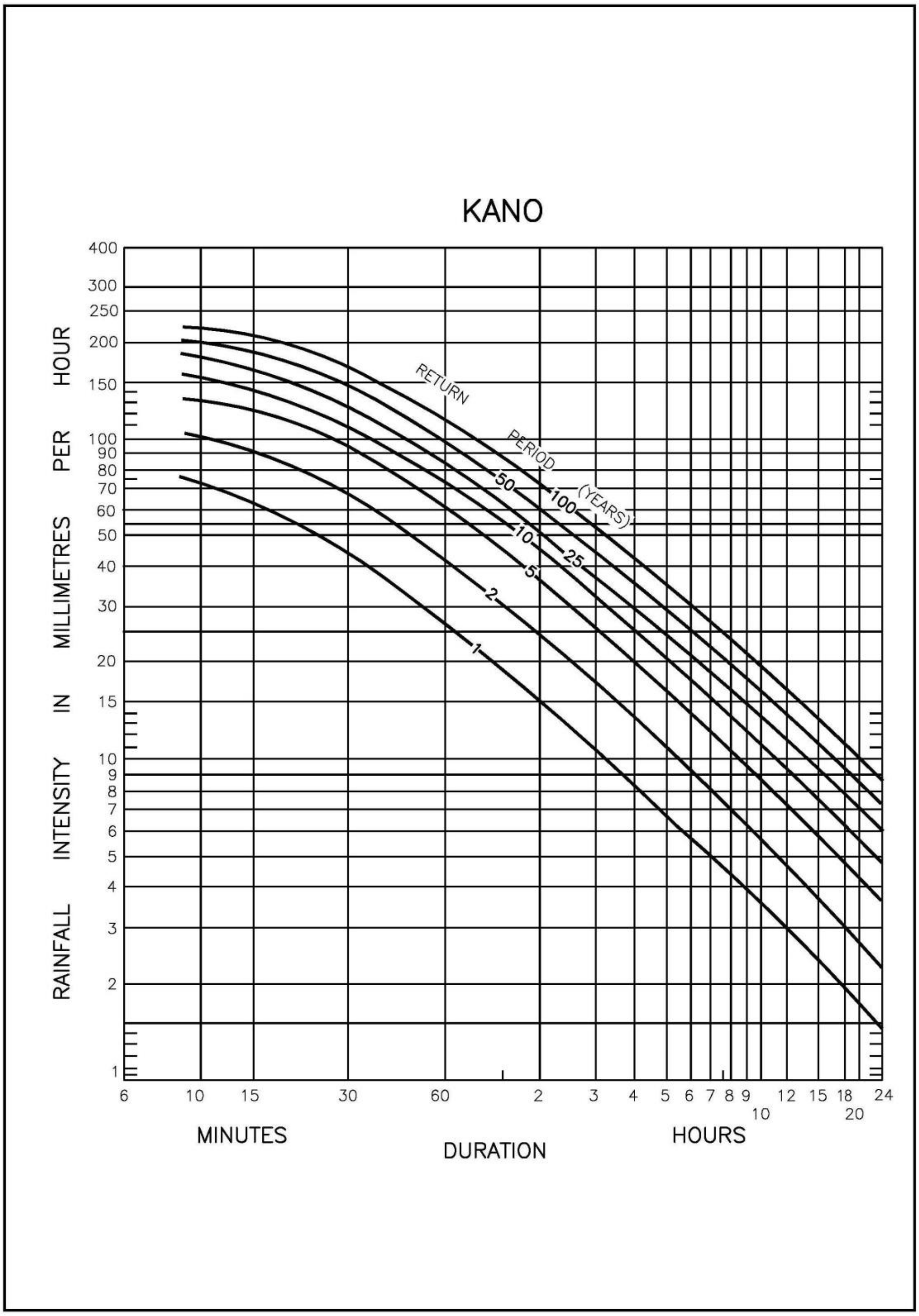


Figure 9.15: Intensity-Duration-Frequency Curves for Kano

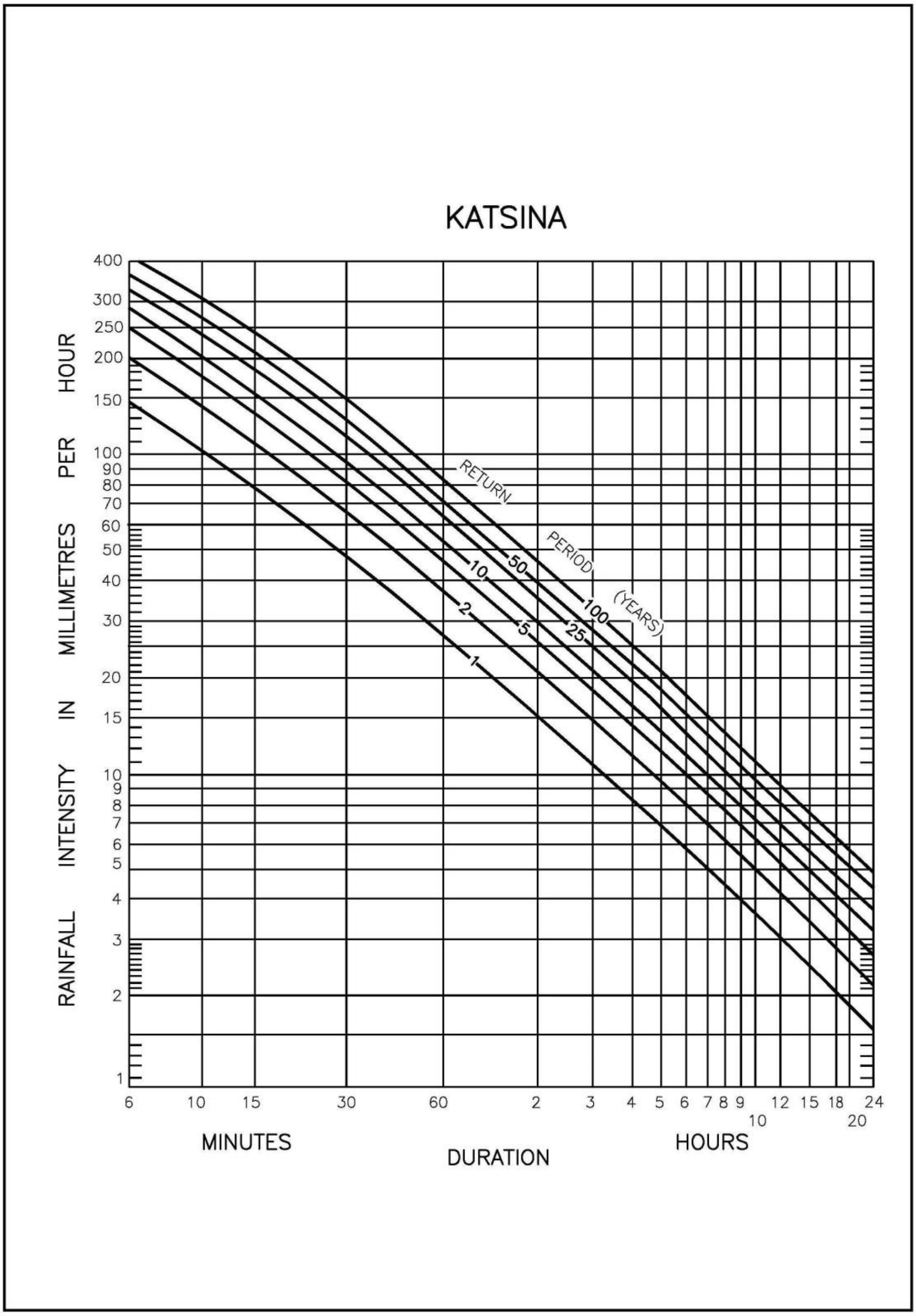


Figure 9.16: Intensity-Duration-Frequency Curves for Katsina

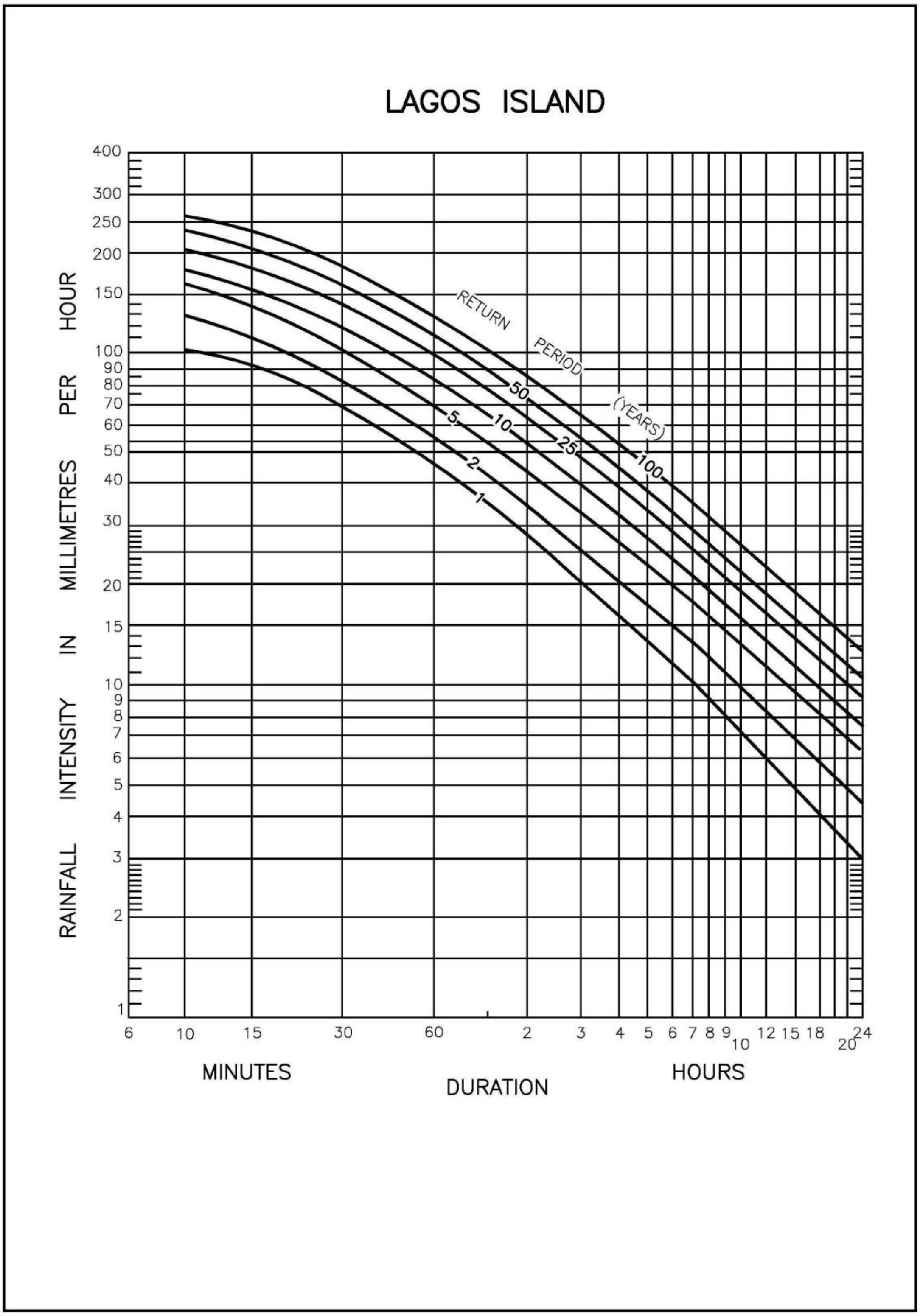


Figure 9.17: Intensity-Duration-Frequency Curves for Lagos Island

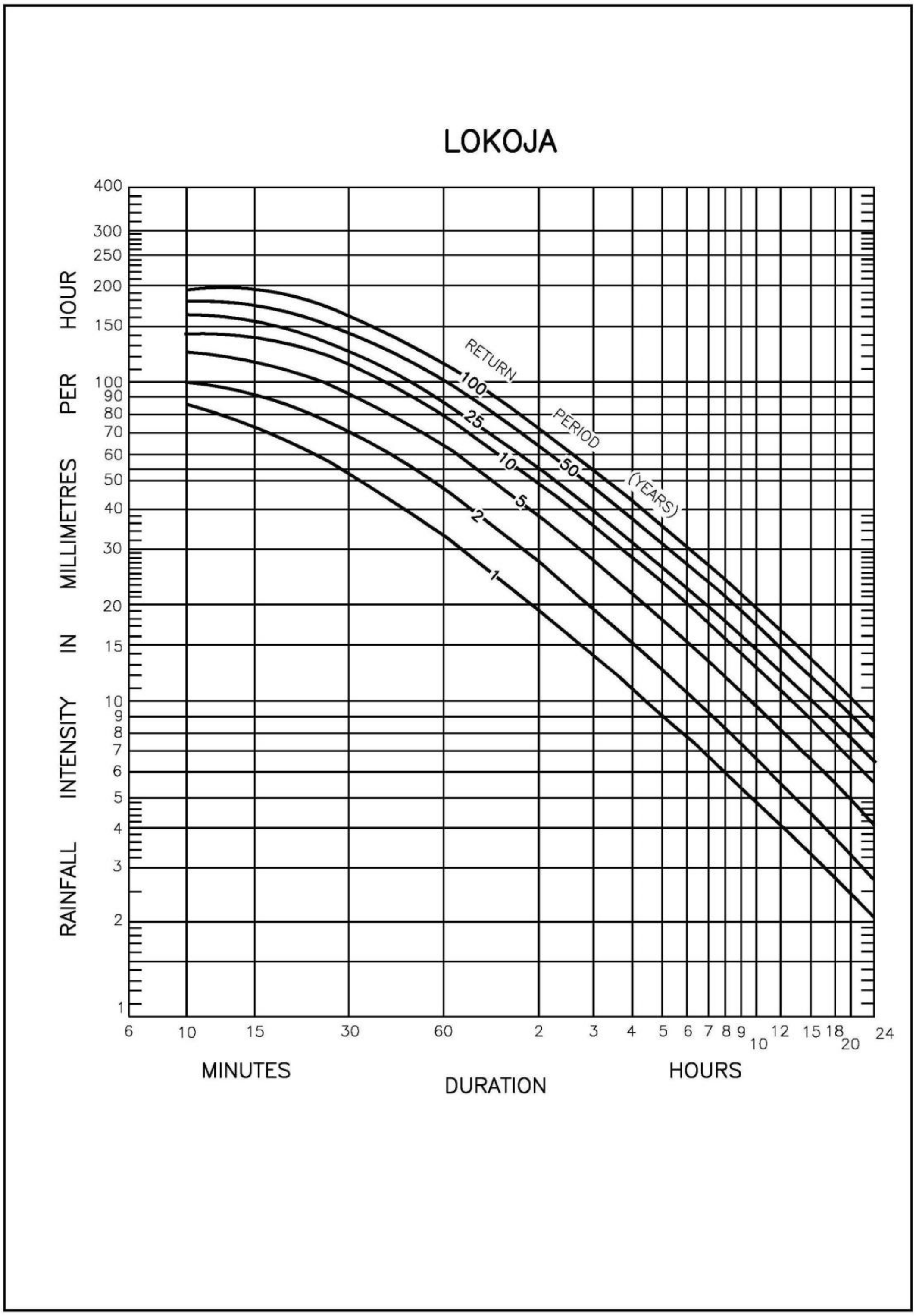


Figure 9.18: Intensity-Duration-Frequency Curves for Lokoja

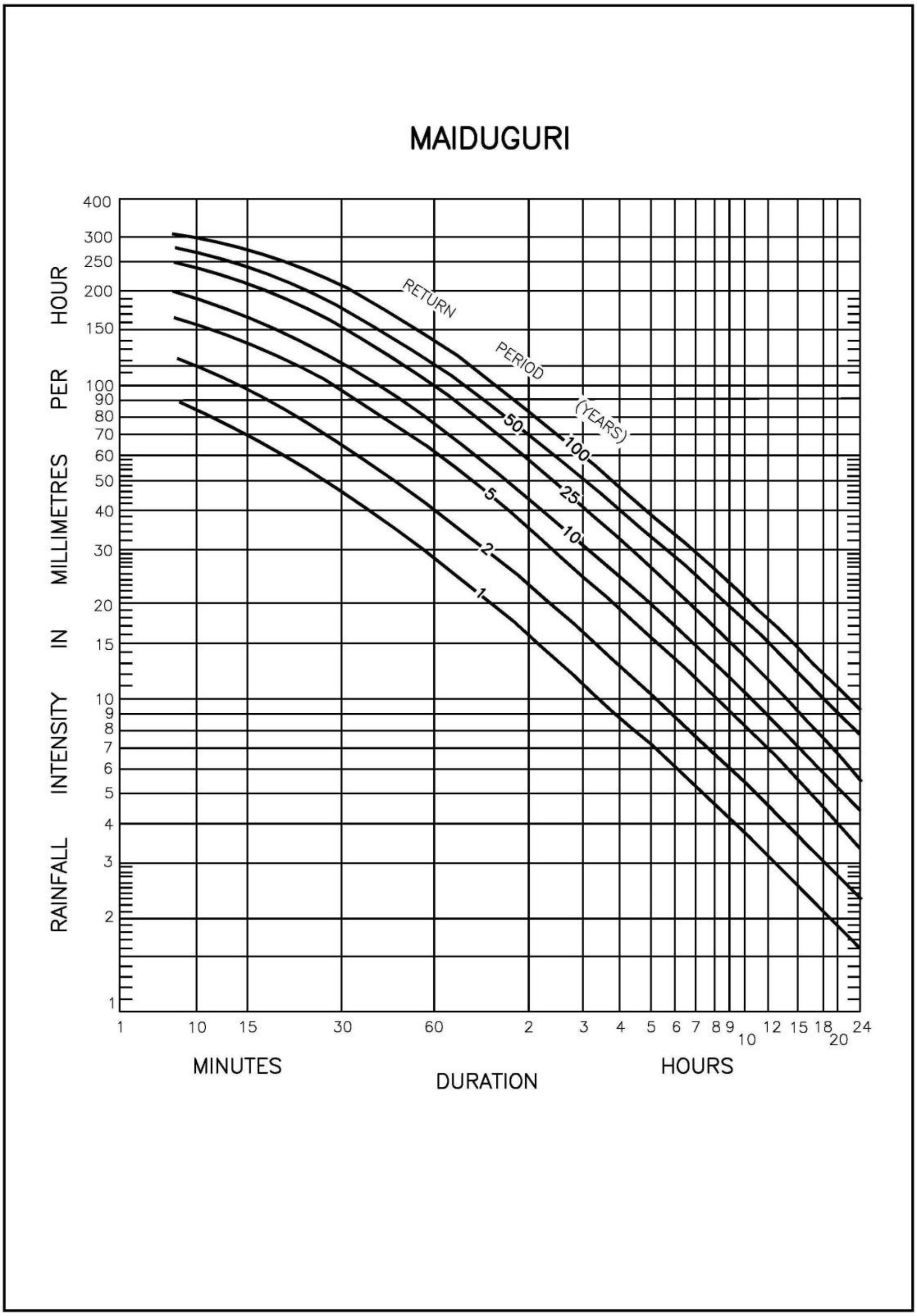


Figure 9.19: Intensity-Duration-Frequency Curves for Maiduguri

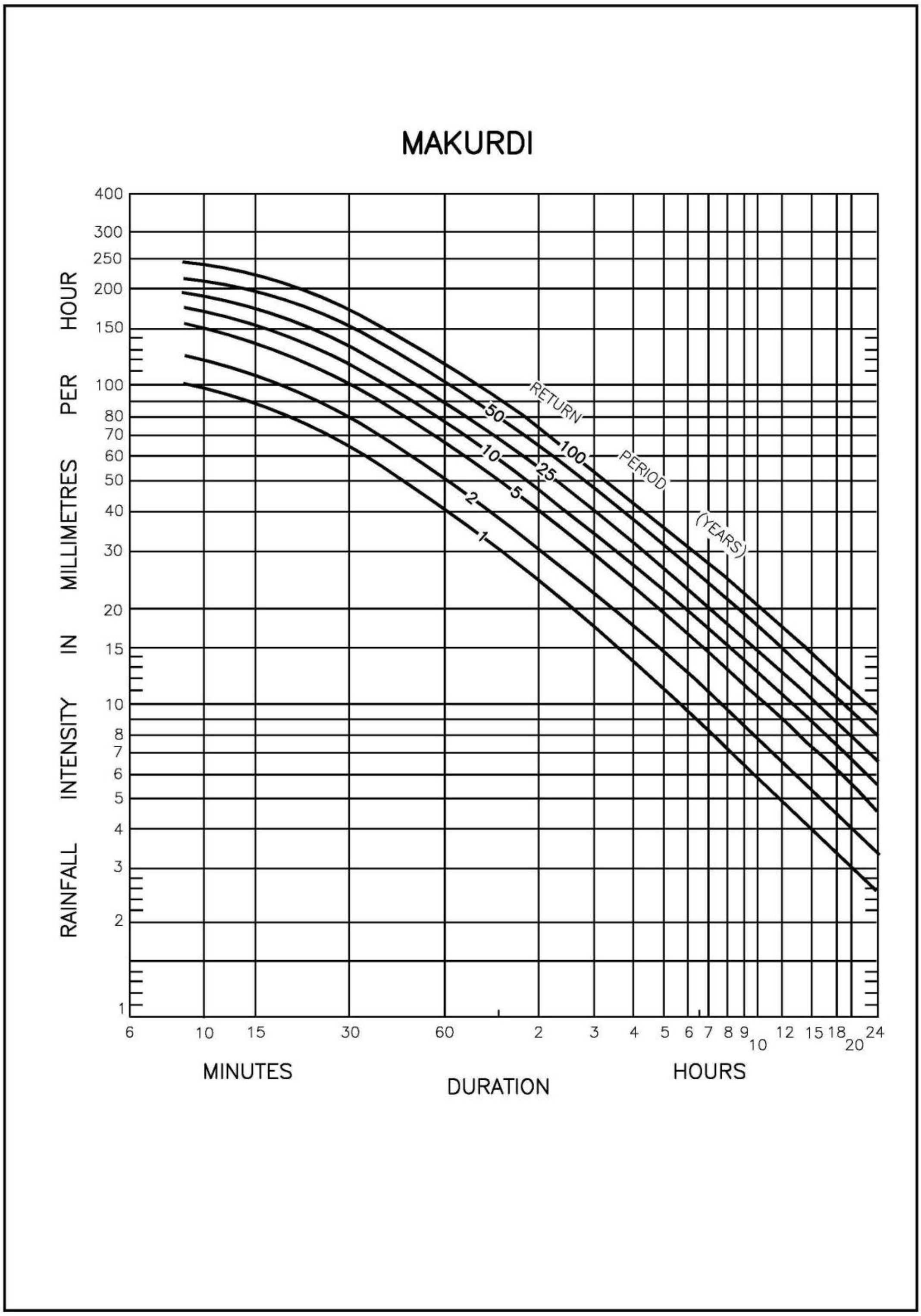


Figure 9.20: Intensity-Duration-Frequency Curves for Makurdi

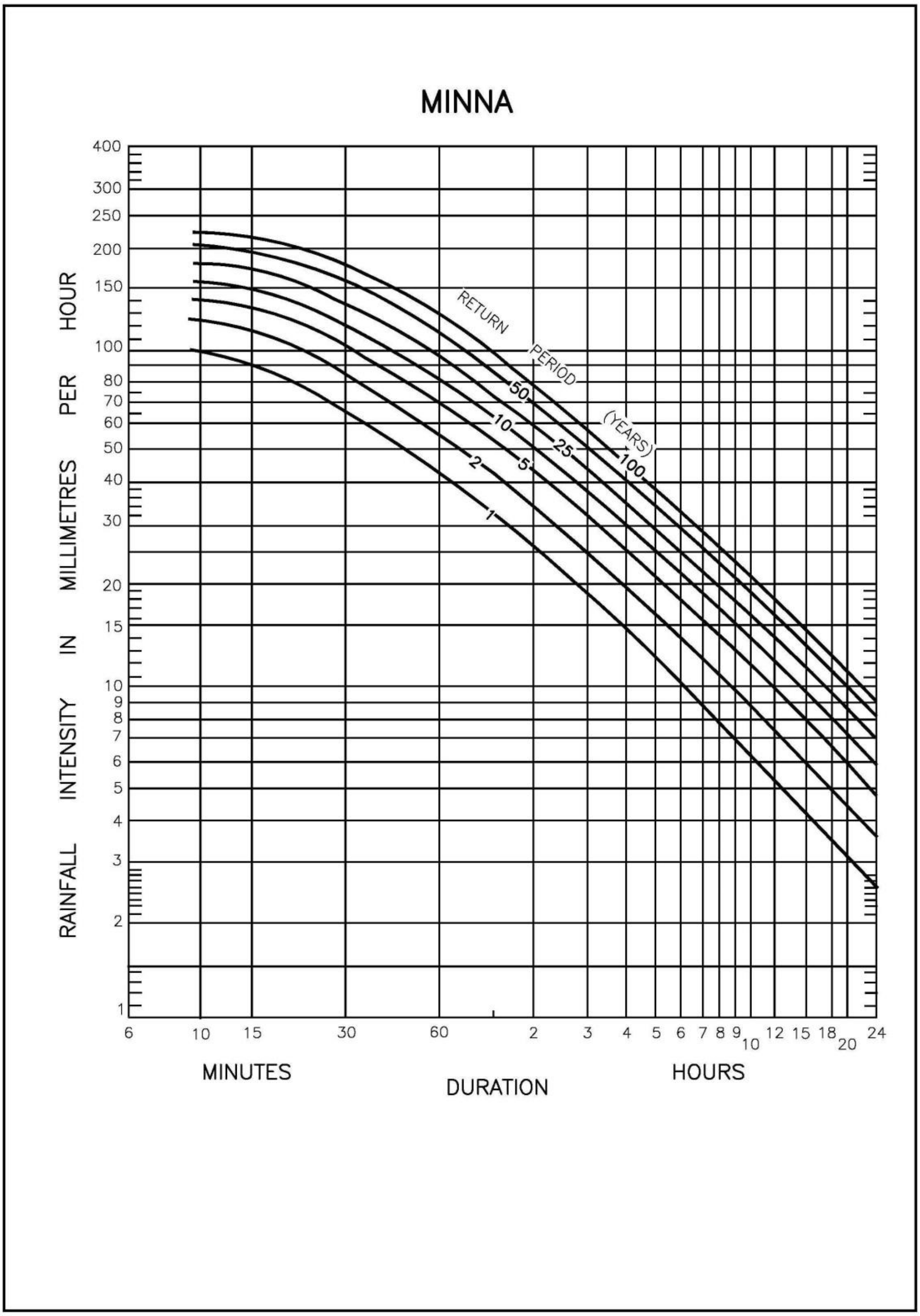


Figure 9.21: Intensity-Duration-Frequency Curves for Minna

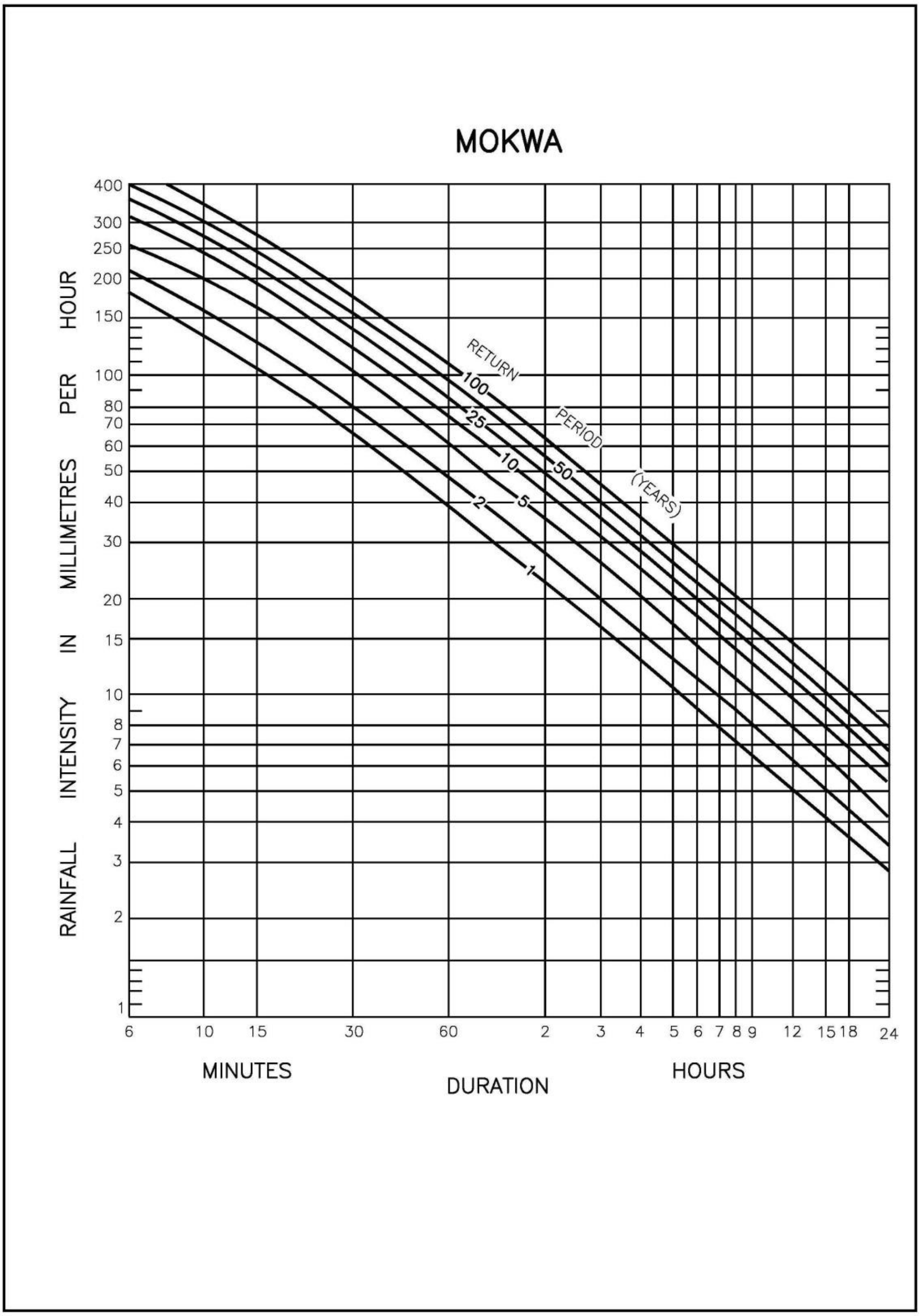


Figure 9.22: Intensity-Duration-Frequency Curves for Mokwa

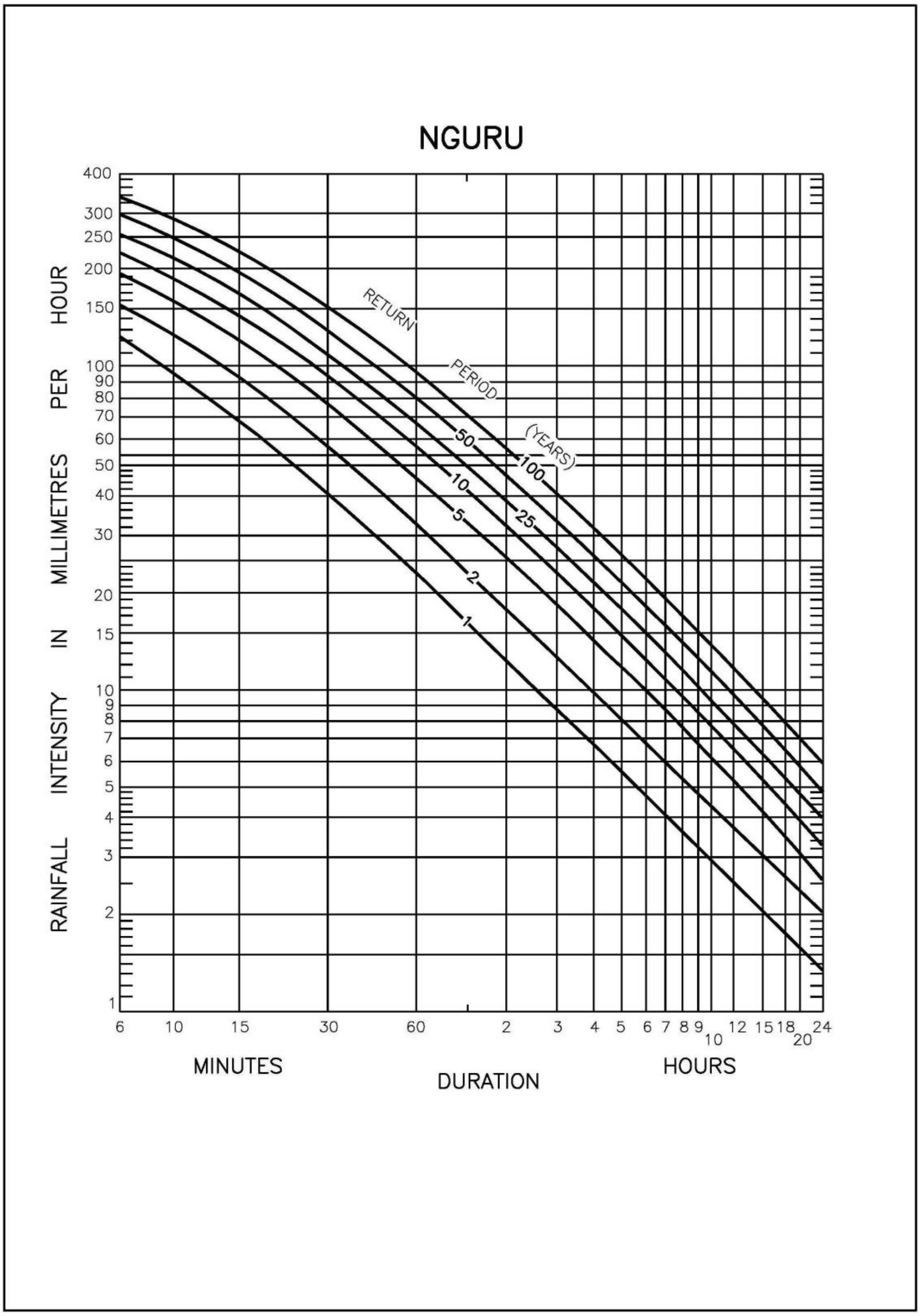


Figure 9.23: Intensity-Duration-Frequency Curves for Nguru

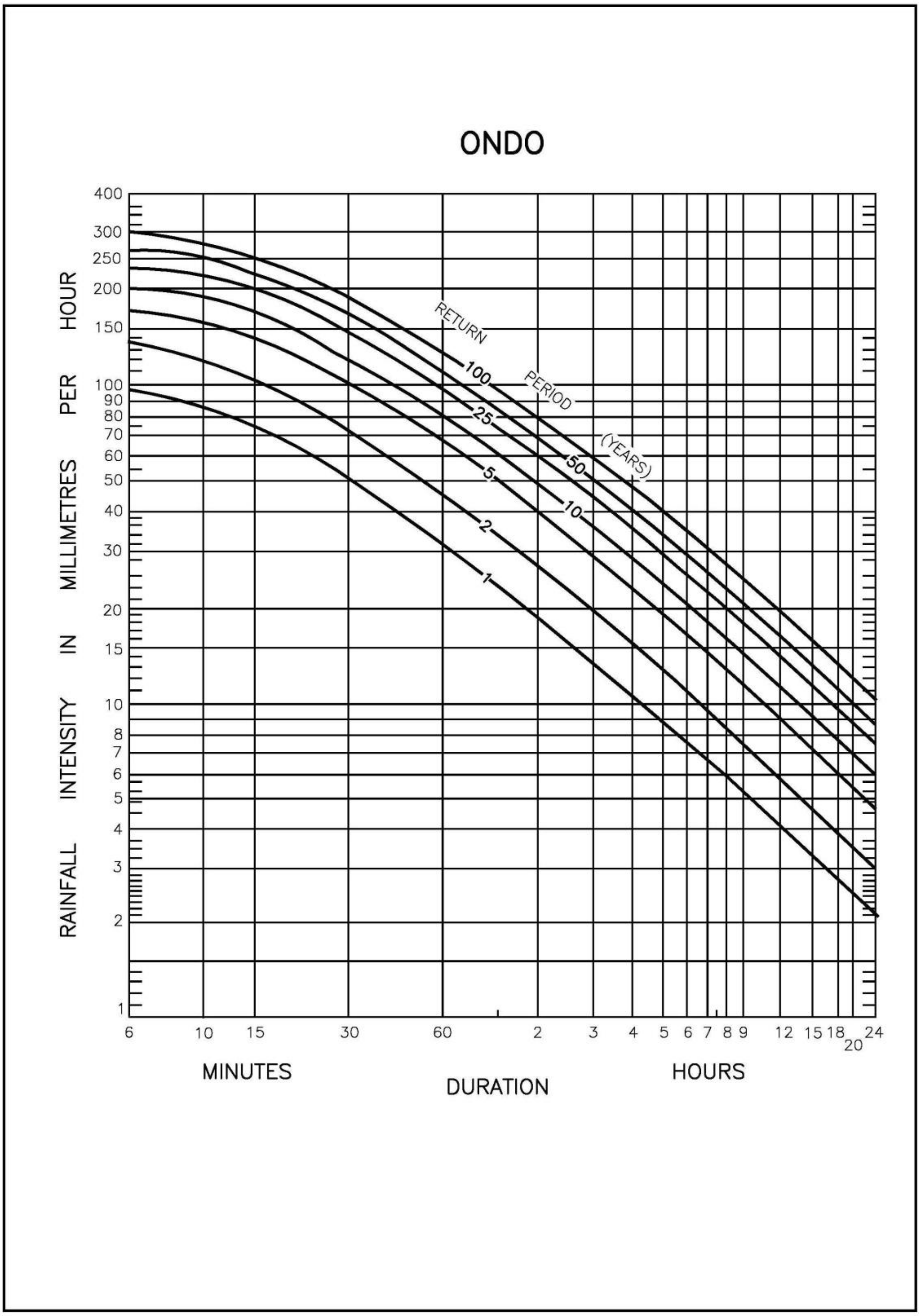


Figure 9.24: Intensity-Duration-Frequency Curves for Ondo

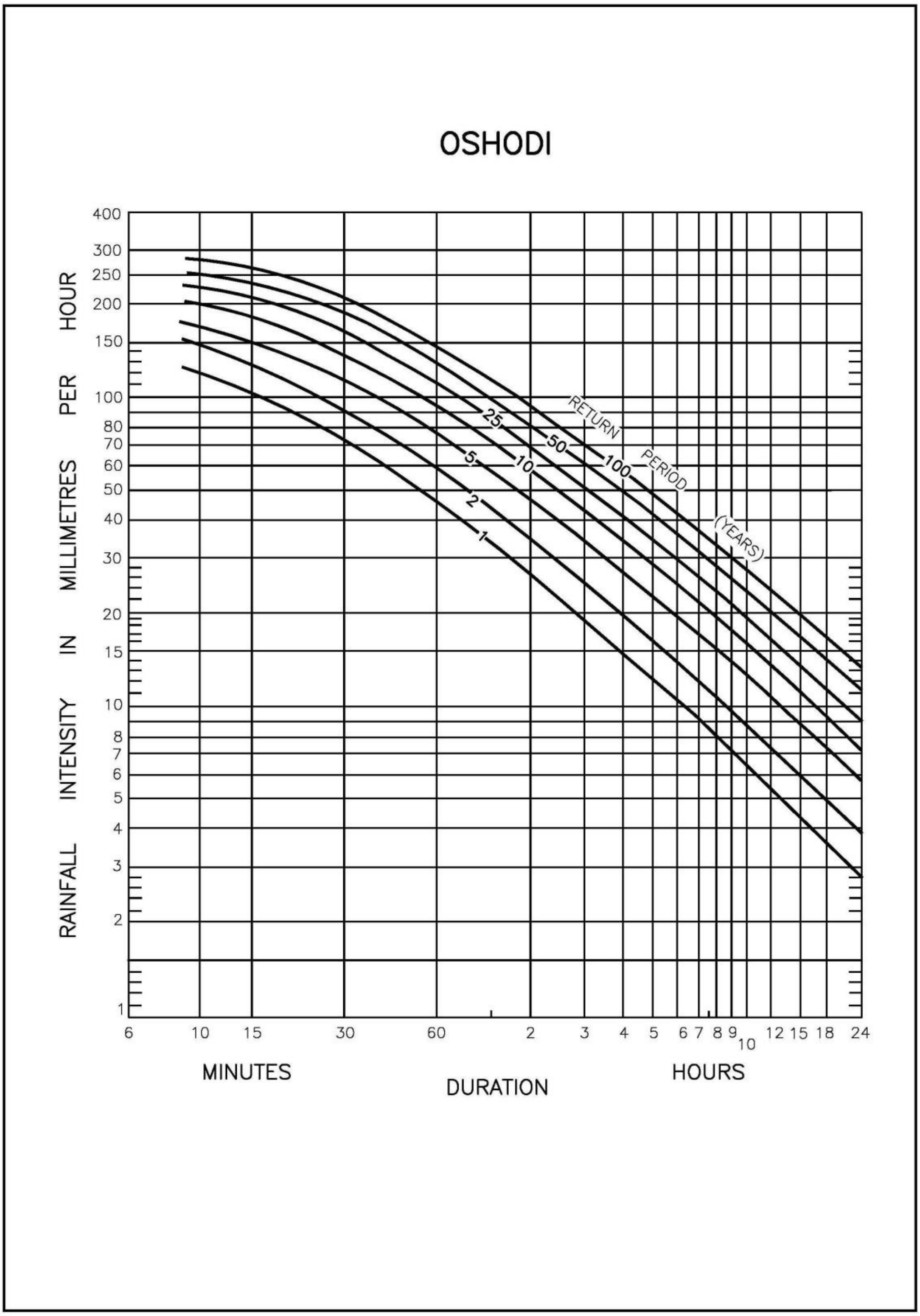


Figure 9.25: Intensity-Duration-Frequency Curves for Oshodi

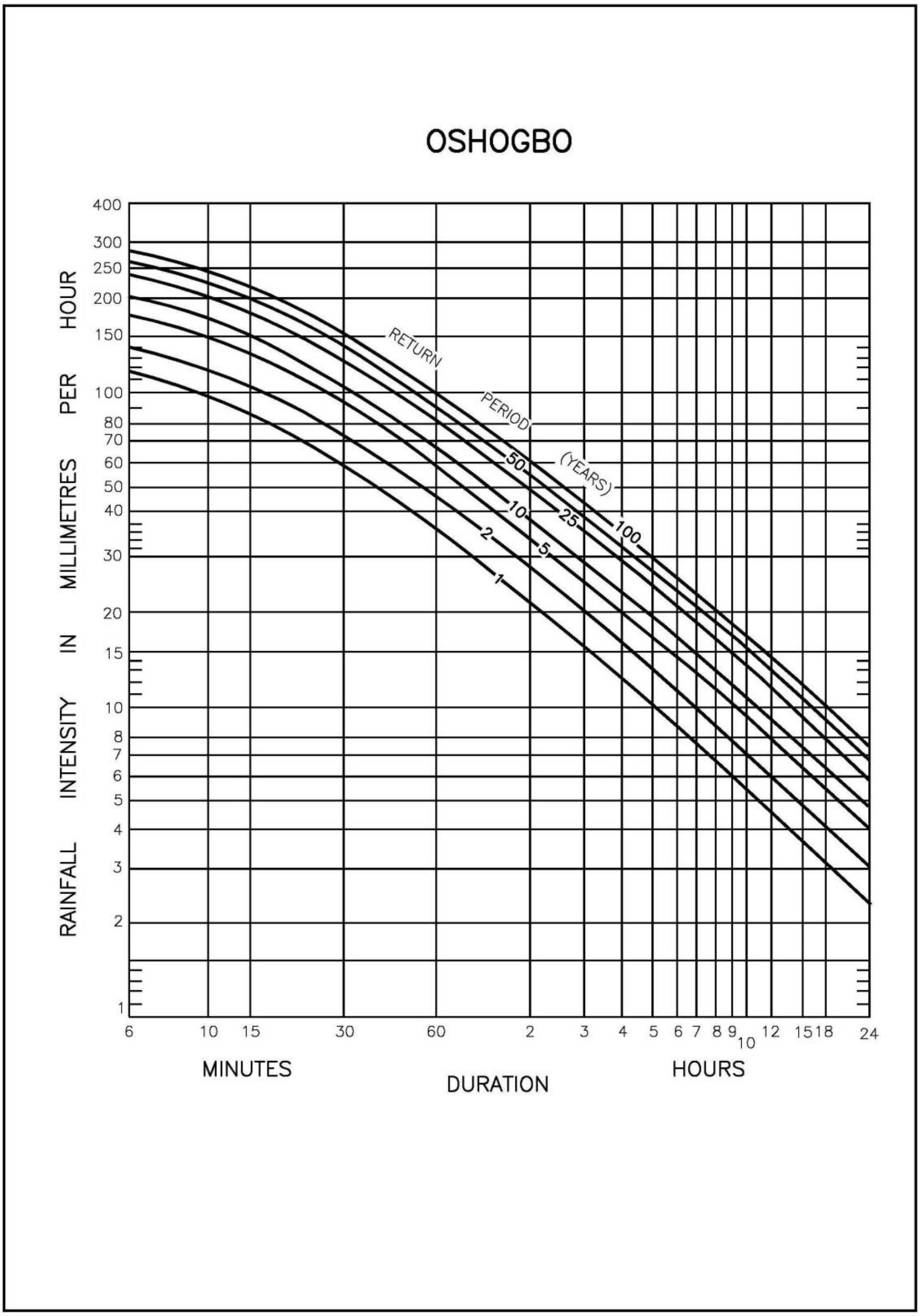


Figure 9.26: Intensity-Duration-Frequency Curves for Oshogbo

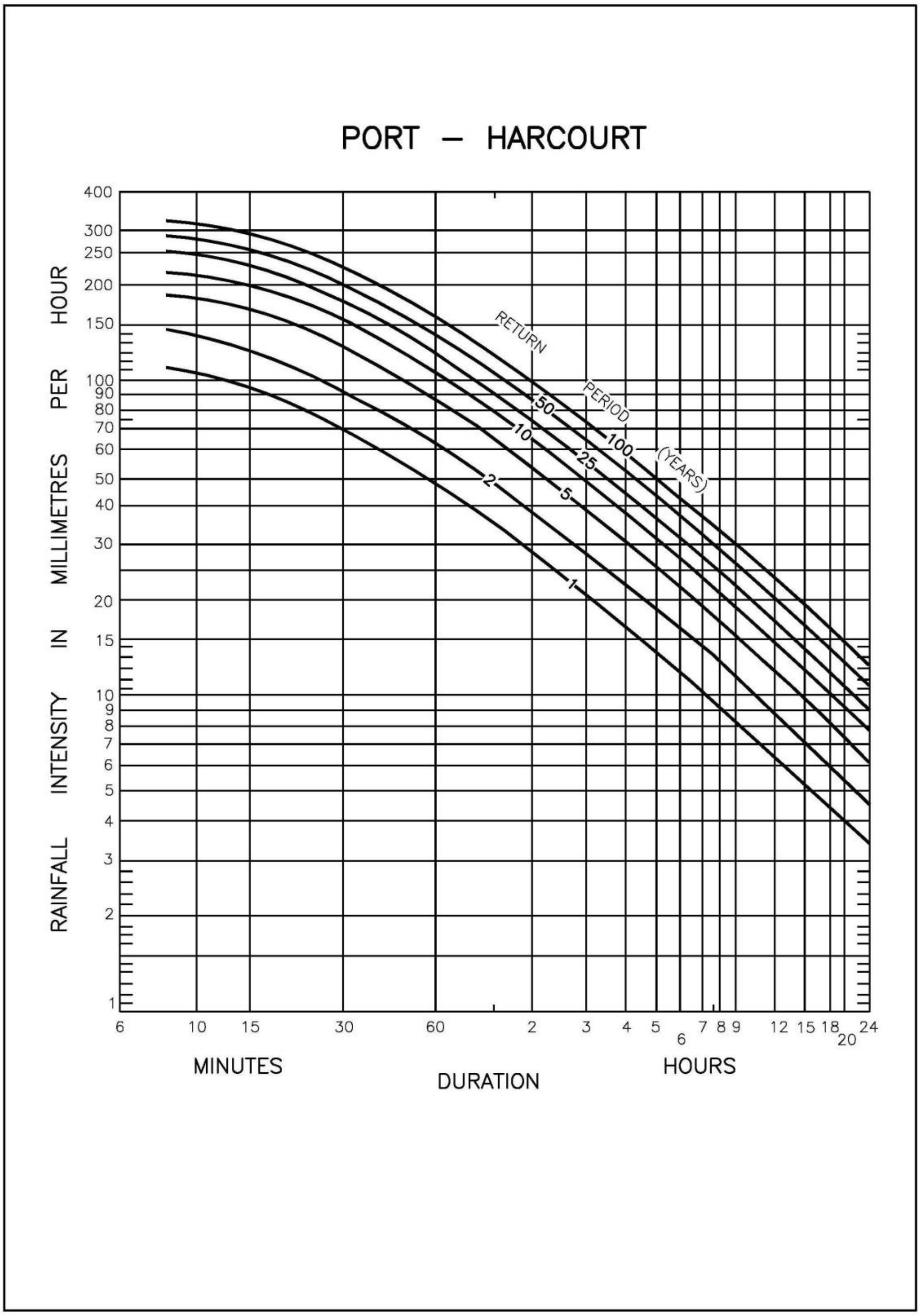


Figure 9.27: Intensity-Duration-Frequency Curves for Port Harcourt

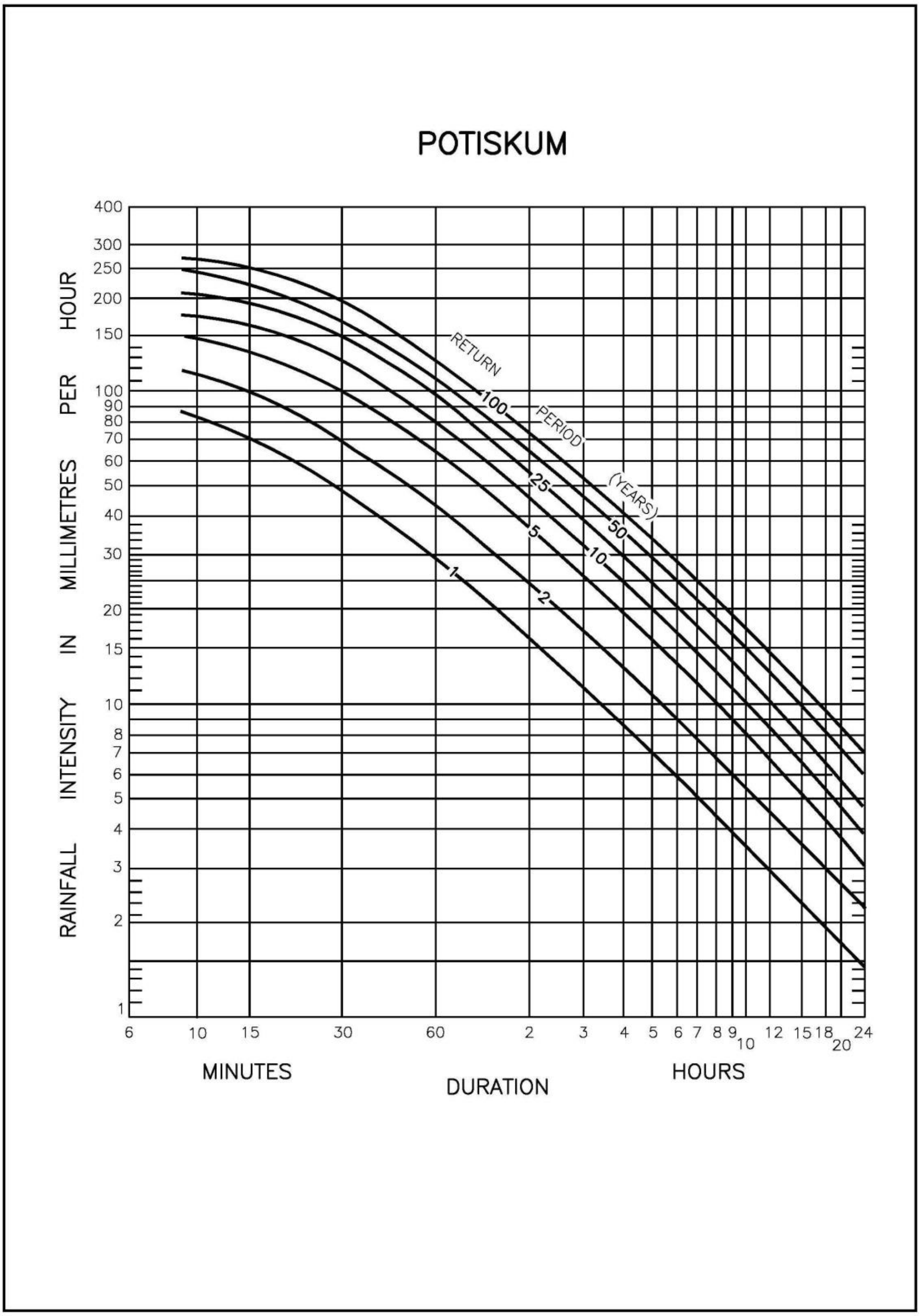


Figure 9.28: Intensity-Duration-Frequency Curves for Potiskum

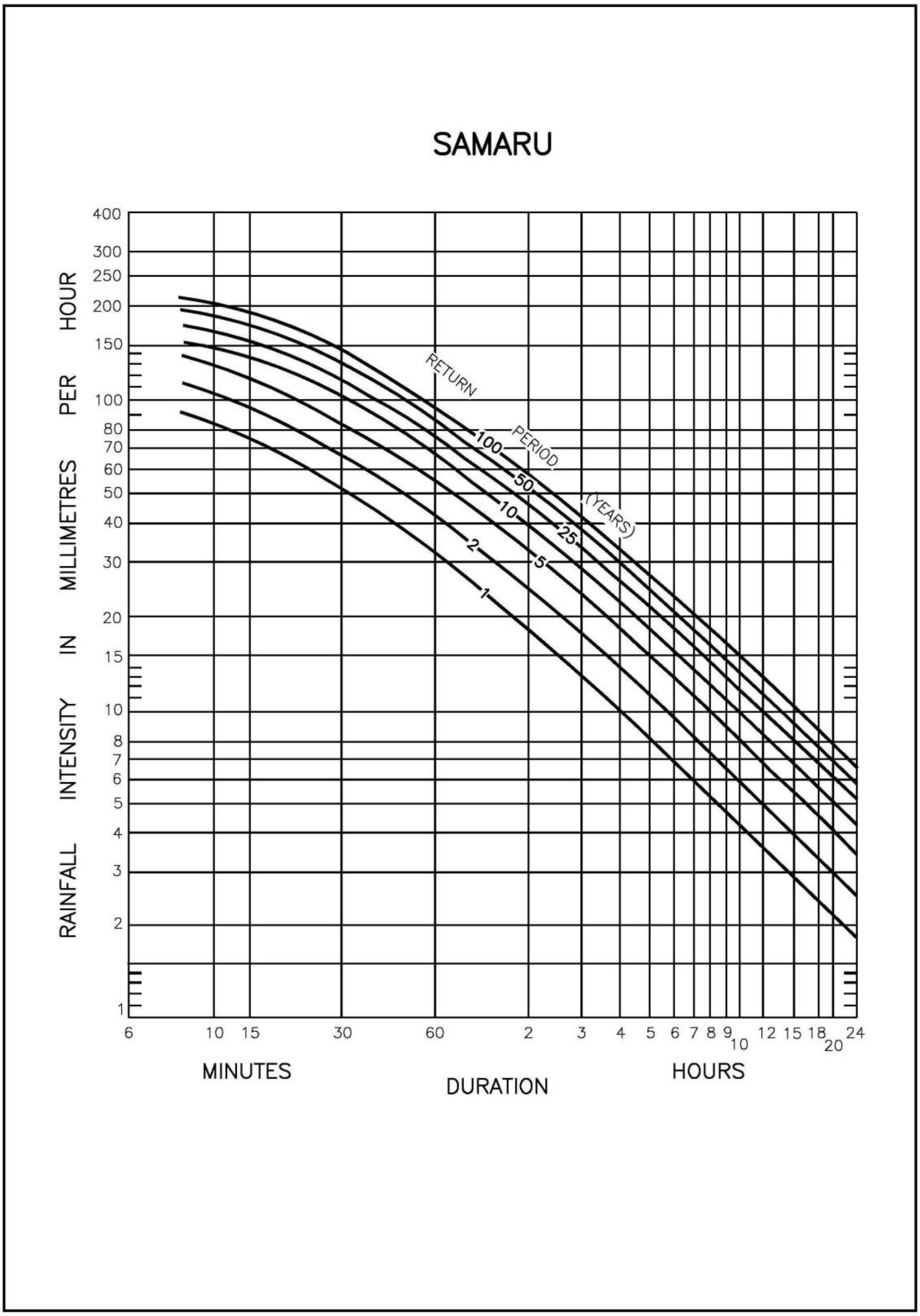


Figure 9.29: Intensity-Duration-Frequency Curves for Samaru

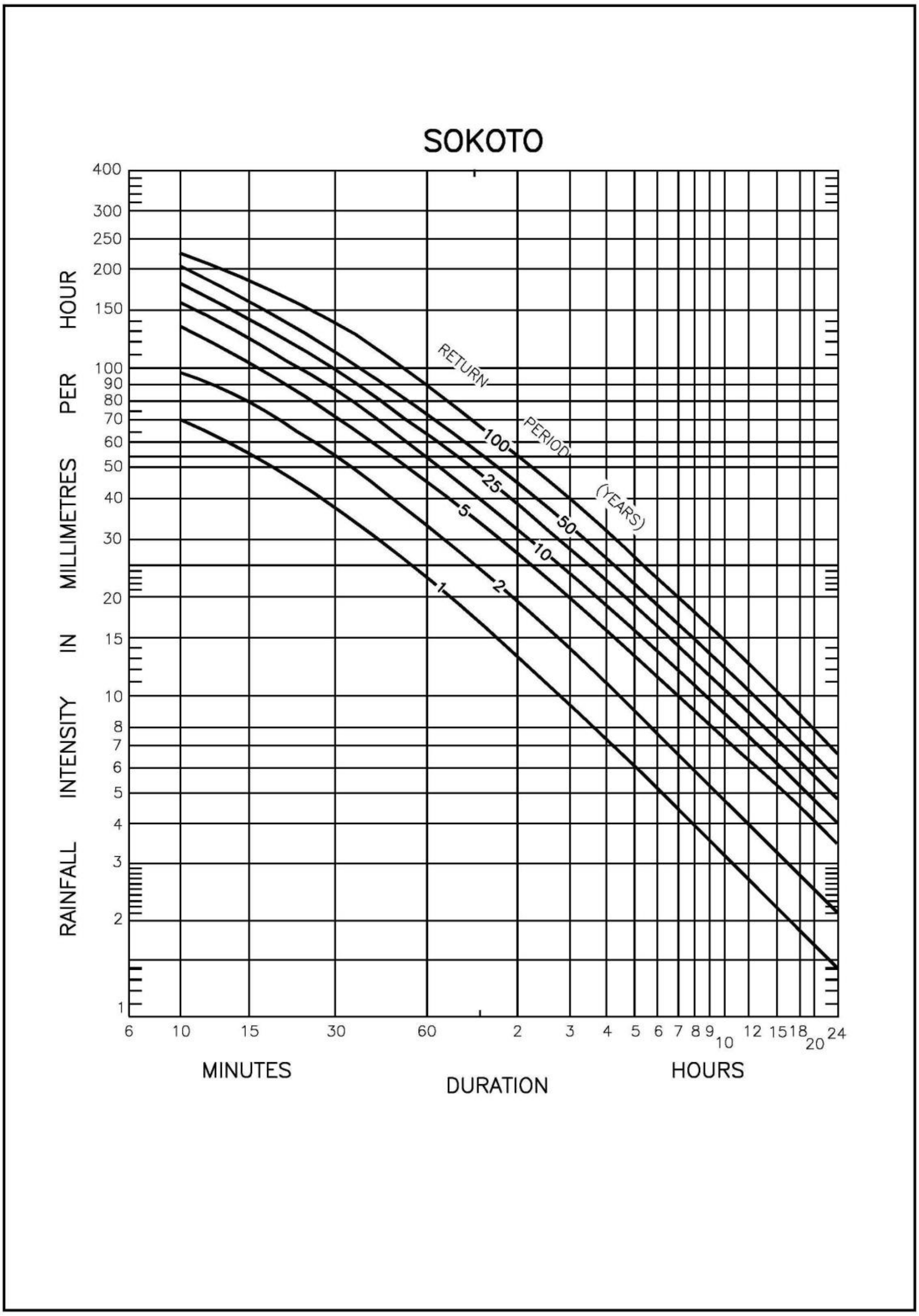


Figure 9.30: Intensity-Duration-Frequency Curves for Sokoto

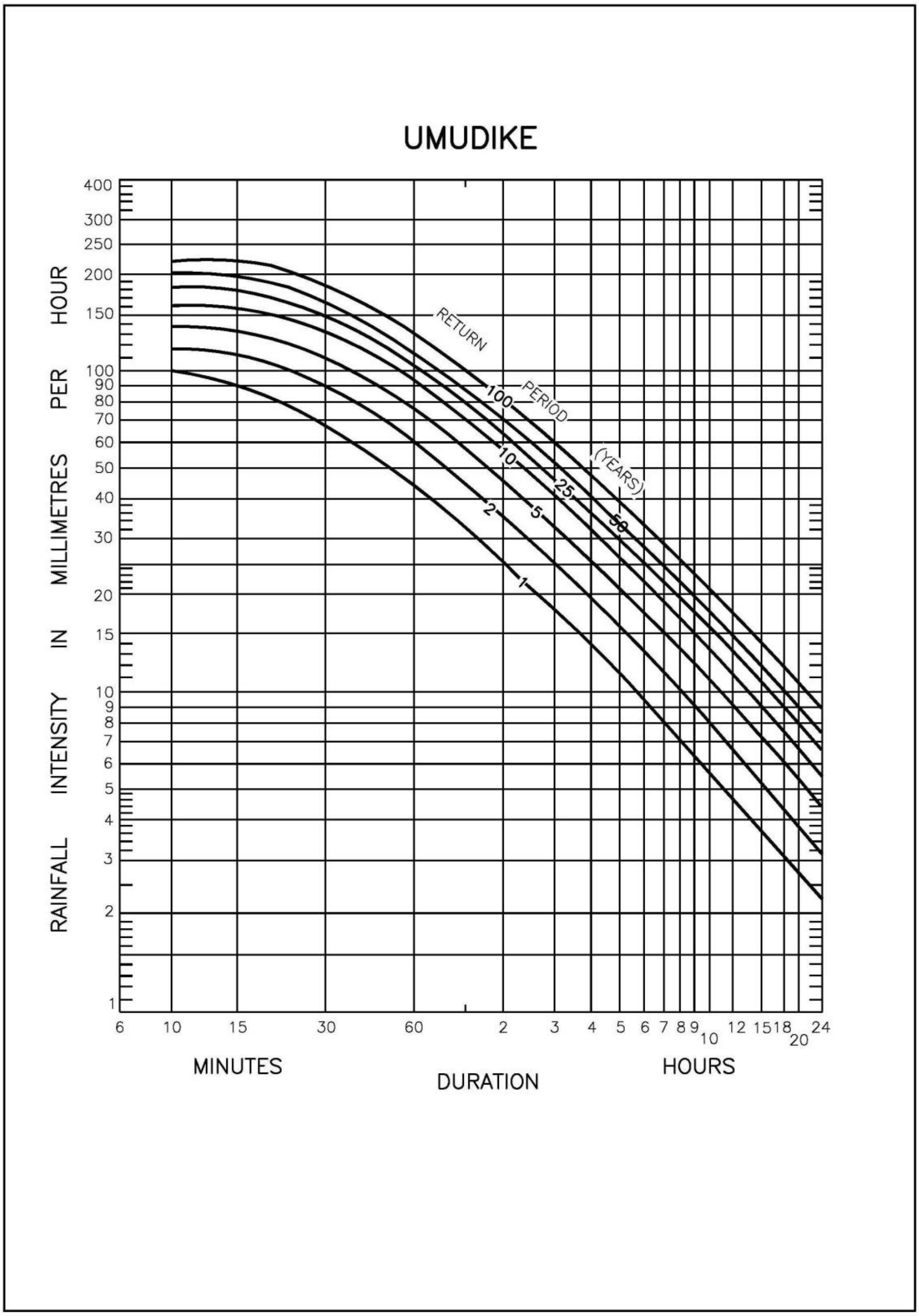


Figure 9.31: Intensity-Duration-Frequency Curves for Umudike

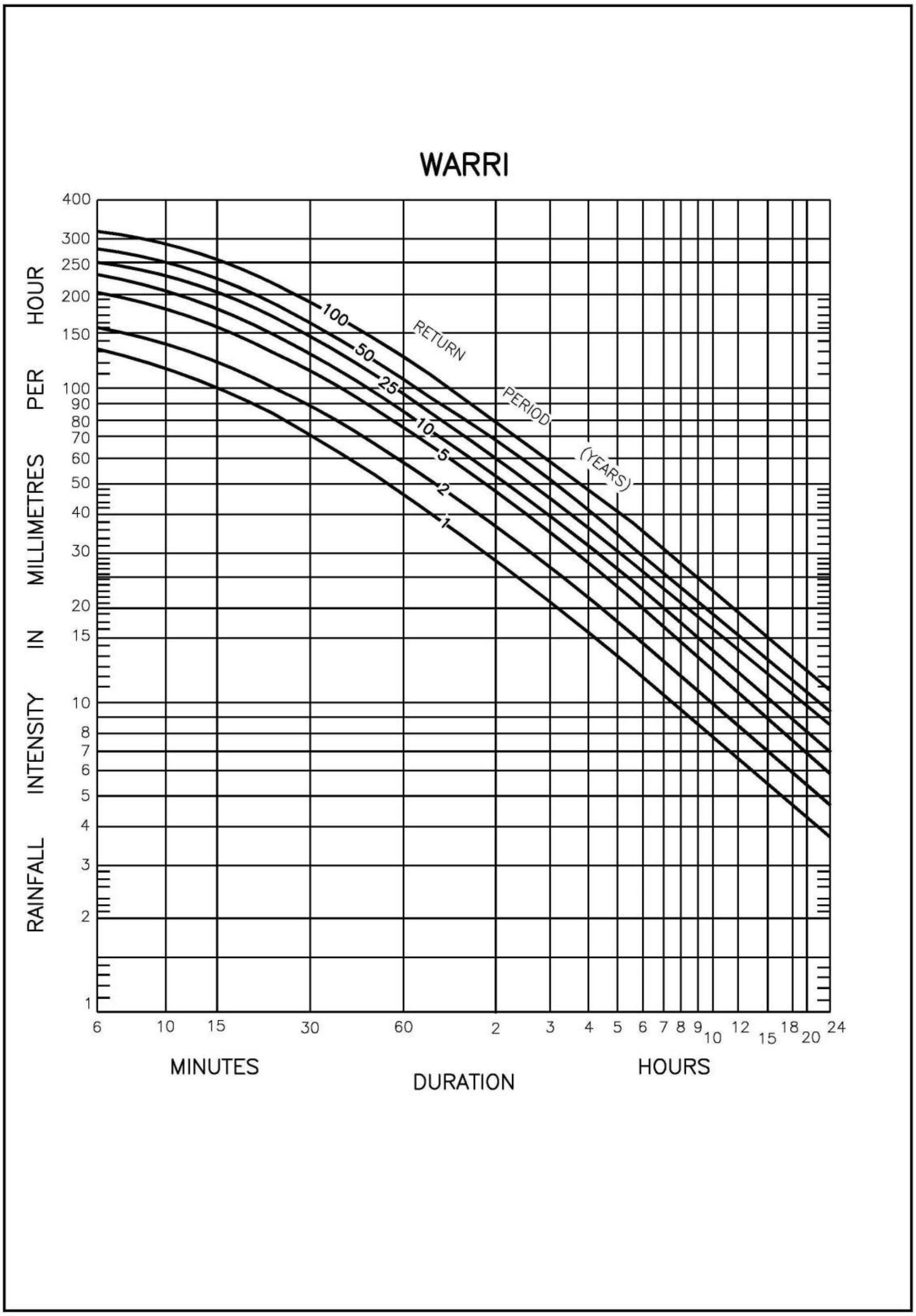


Figure 9.32: Intensity-Duration-Frequency Curves for Warri

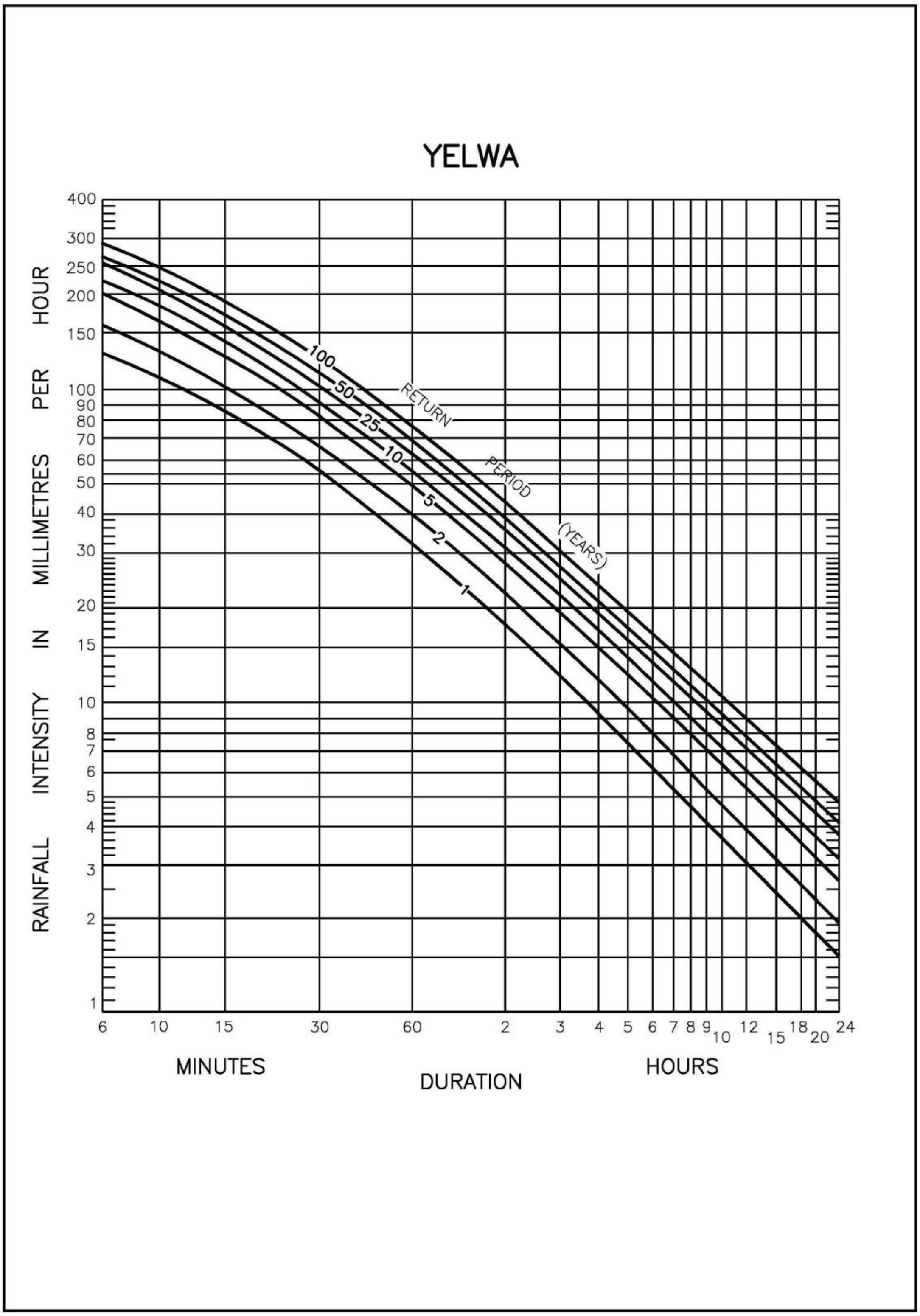


Figure 9.33: Intensity-Duration-Frequency Curves for Yelwa

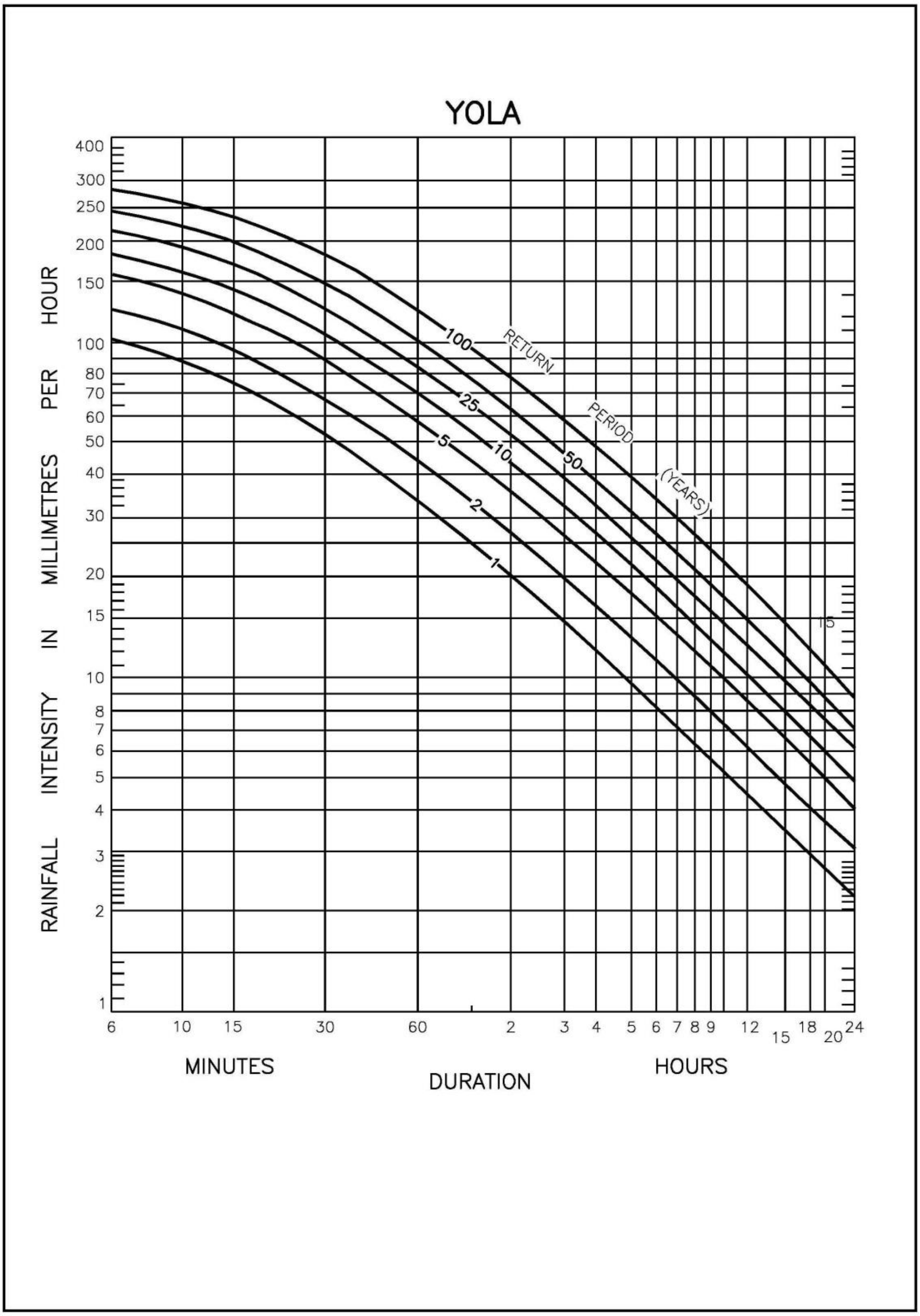


Figure 9.34: Intensity-Duration-Frequency Curves for Yola

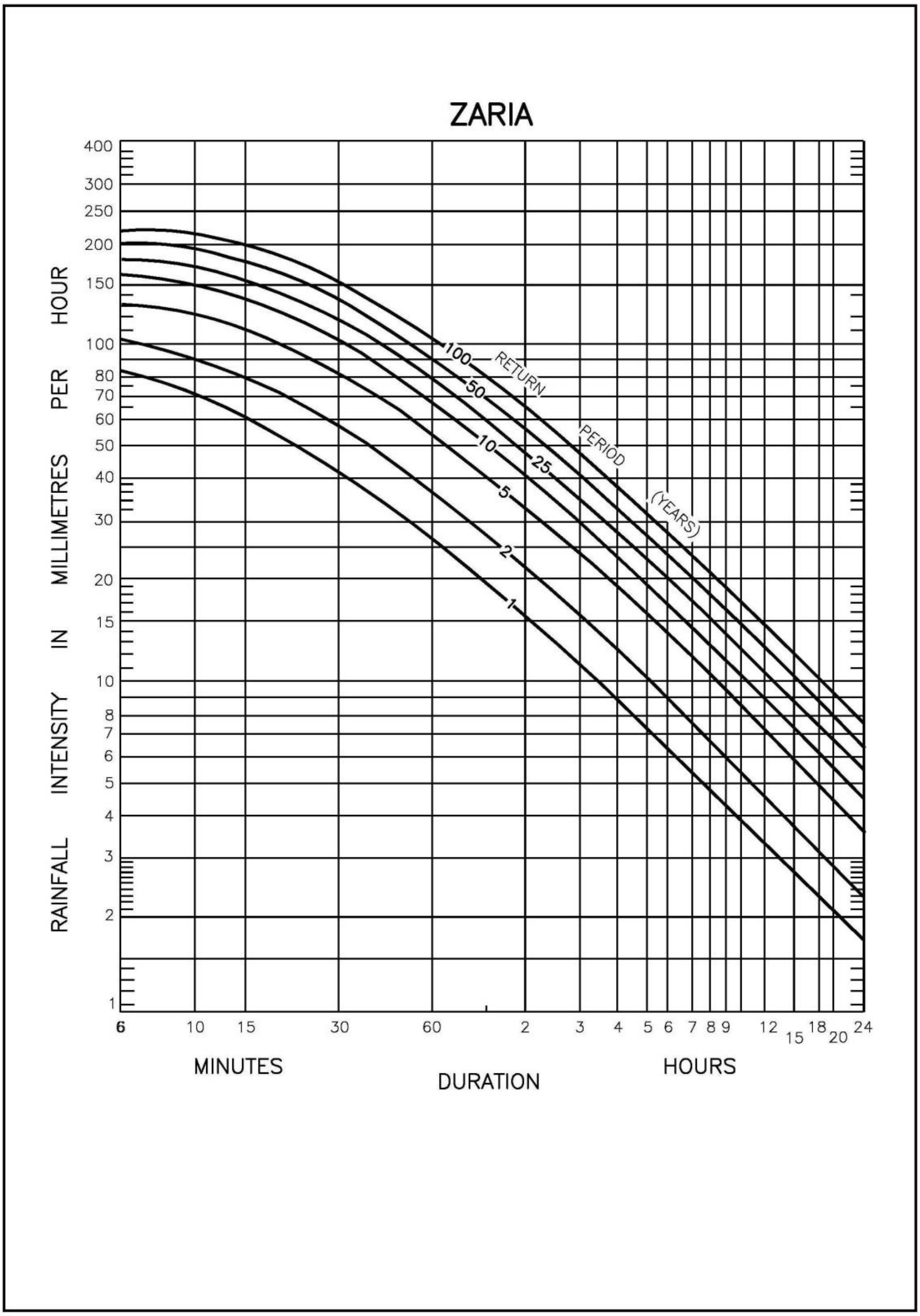


Figure 9.35: Intensity-Duration-Frequency Curves for Zaria

Table 9.1: Mean Monthly Rainfall in mm (1995 to 2005)

Station	North	East	Altitude	No of	Month												
	Lat	Long	metre	years	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Abeokouta	7.11	3.2	66	21	5	31	63	131	135	164	180	110	204	114	10	11	1158
Bauchi	10.18	9.5	599	21	0	0.11	5.2	33	103	160	231	280	173	34	0.13	0	1019.4
Benin City	6.18	5.38	77	21	17	40	112	188	225	262	343	320	315	262	62	27	2171
Enugu	6.27	7.29	224	21	5.7	7.6	61.2	170	231	263	268	253	271	196	19	2	1747.5
IKEJA	0.635	3.2	38	21	11	43	94	166	204	273	211	114	202	164	72	28	1582
Ilorin	8.26	4.3	361	21	8	11	45	93	161	185	148	147	222	124	11	5	1160
Jos	9.52	8.54	1269	21	0.3	3	27	105	180	223	258	260	251	51	0.2	0	1358.5
Kaduna	10.36	7.27	635	21	0	0.6	10.3	44	100.6	181.5	236.2	311	230	77.9	0	0	1192.1
KANO	12.02	8.32	465	21	0	0.4	1.14	25	70	277	281	342	157	15	0.3	0	1168.8
Katsina	13.01	7.41	509	21	0	0	1.6	14.3	33.6	67.8	153.3	143.6	75.5	14.1	0	0	503.8
LAGOS	6.77	3.24	3	21	18	35	73	157	242	375	222	134	220	200	38	13	1727
Lokoja	7.48	6.44	41	21	3	18	38	107	152	192	174	207	219	116	4	0.2	1230.2
Maiduguri	11.51	13.05	405	21	0	0	0	9	31	71	166	199	109	12	0	0	597
Makurdi	7.44	8.32	110	21	2	2	18	79	116	230	166	216	218	90	10	1	1148
Minna	9.37	6.32	254	21	0.5	0.6	10	66	144	180	202	260	220	105	0.24	0	1188.3
Ngen	10.5	12.8	270	21	0	0	0.7	2.5	11.8	38.8	114	242	60	62.7	0	0	532.3
Ogaja	6.39	8.42	225	21	8	17	62	114	238	261	249	268	305	240	22	7	1791
Owerri	5.3	7.02	69	21	32	37	121	196	265	352	377	358	389	233	60	11	2431
Shaki	8.69	3.2	285	21	4	15	62	122	152	176	170	186	202	119	5	7	1220
Port Harcourt	4.51	7.01	64	21	28	53	119	142	272	261	370	324	345	256	80	29	2270
Potiskum	11.43	11.03	407	21	0	0.11	10	15.5	35.4	93	184	206	88	19	0	0	642
Sokoto	13.01	5.16	345	21	0	0	41	5.8	51.6	101	195	180	100	9.7	0	0	647.3
Yola	9.13	12.28	173	21	0	0	8	45	123	155	168	190	205	80	5	0	977
Zaria	11.06	7.44	630	21	0	3	3	45	115	160	218	315	218	33	3	0	1113

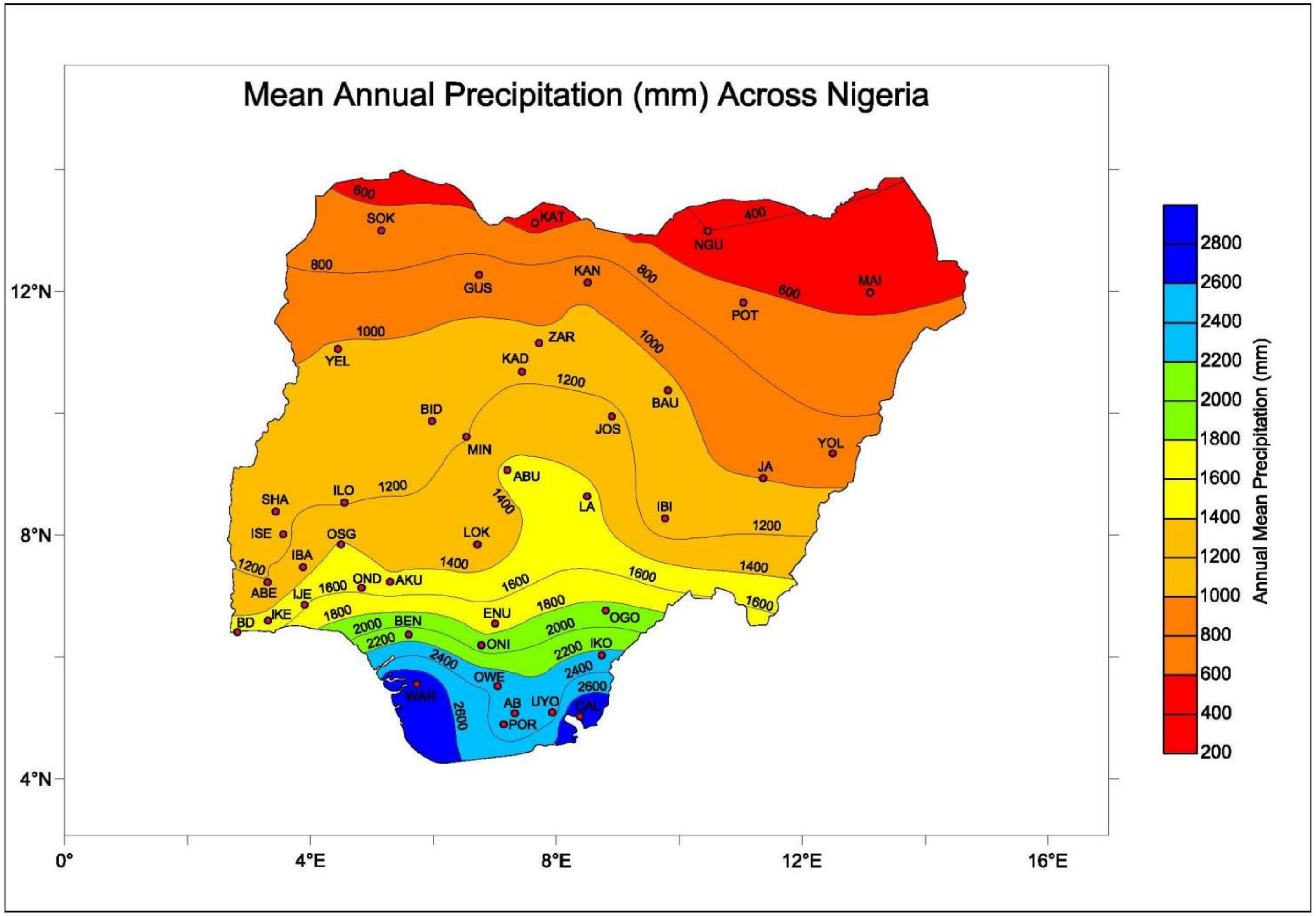


Figure 9.36: Map showing mean annual precipitation for Nigeria

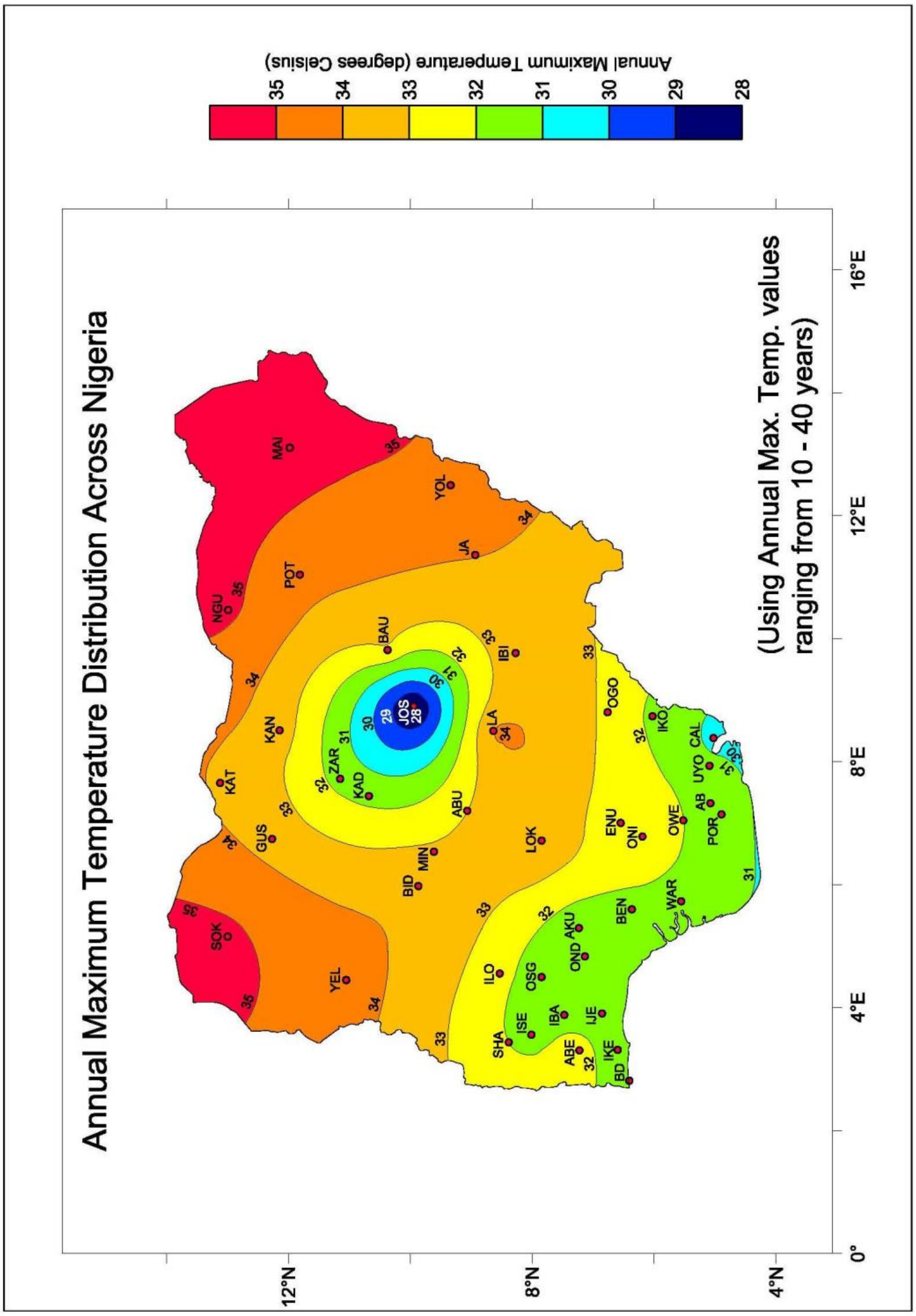


Figure 9.37: Map showing mean annual maximum temperature for Nigeria

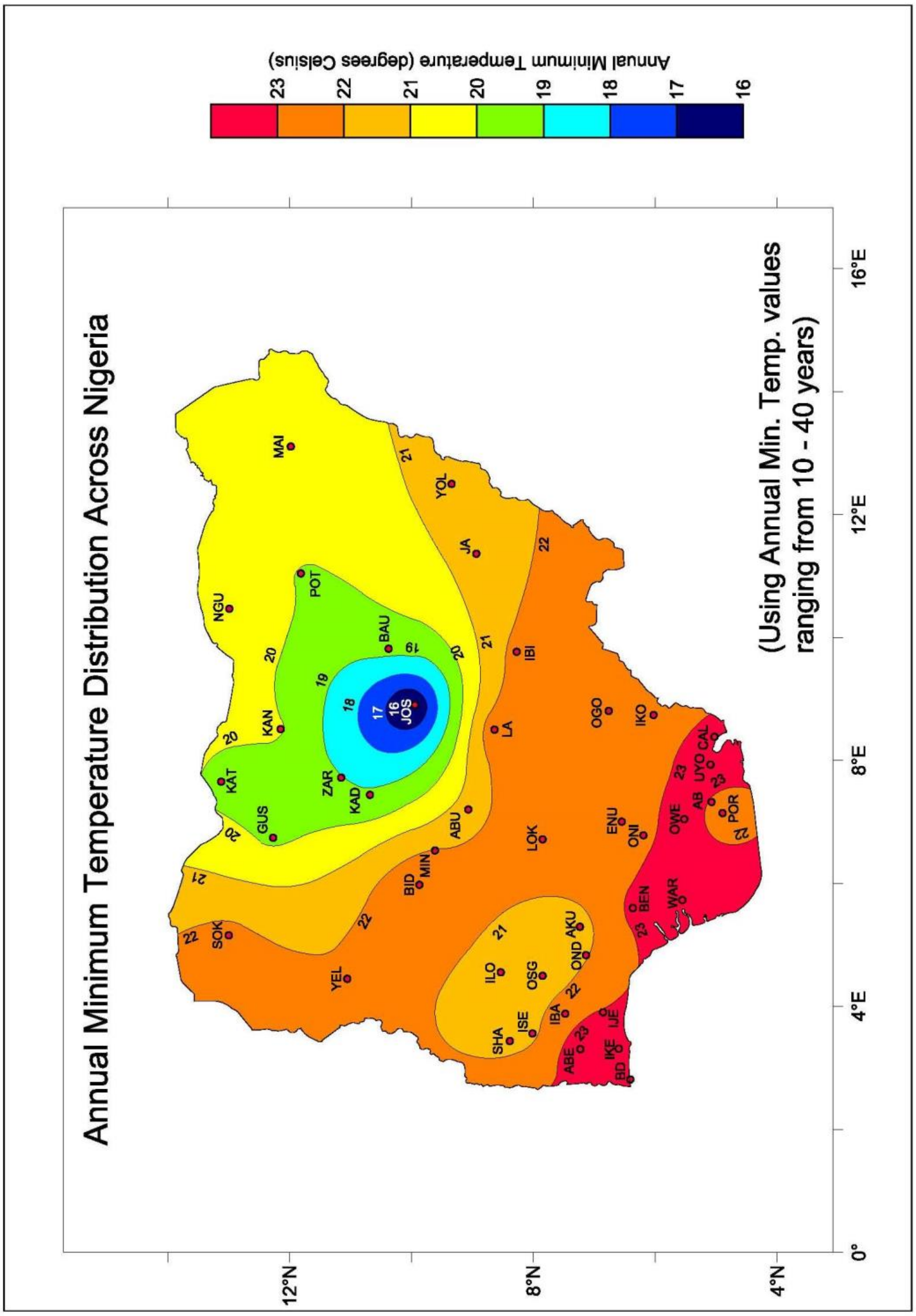


Figure 9.38: Map showing mean annual minimum temperature for Nigeria

10: Appendix 1B: Rainfall IDF Maps of Nigeria

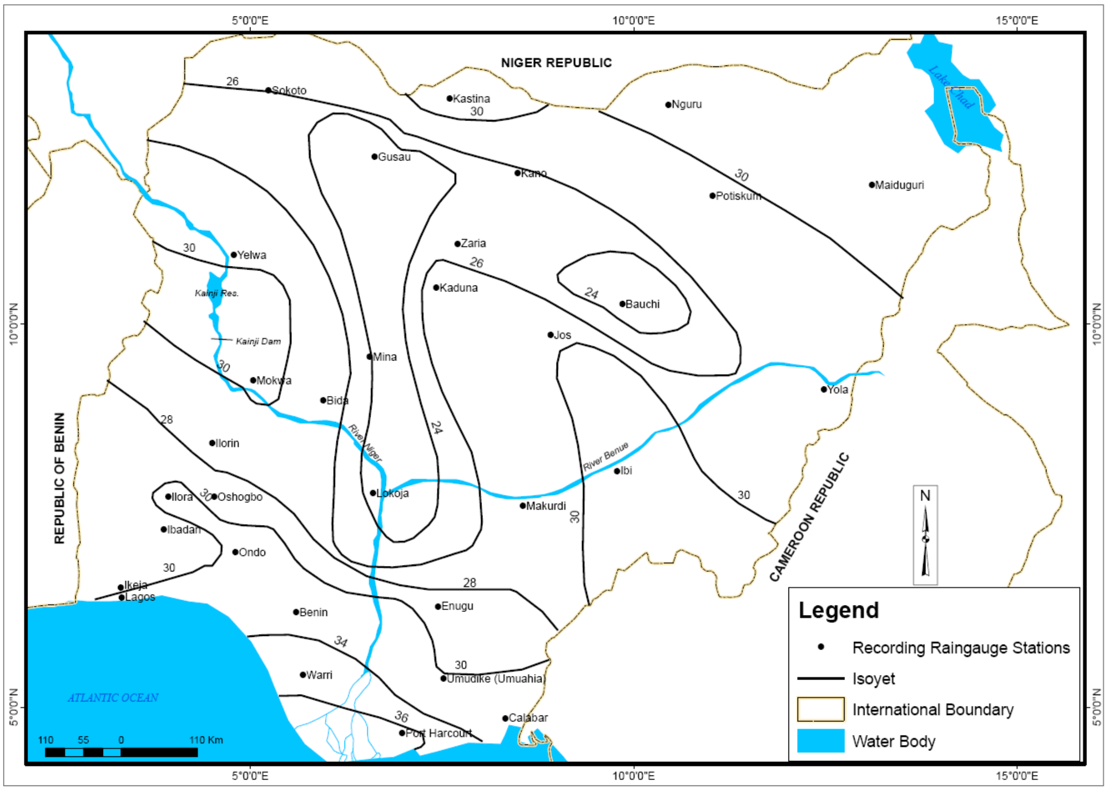


Figure 10.1: 10-year rainfall for duration of 10-minutes

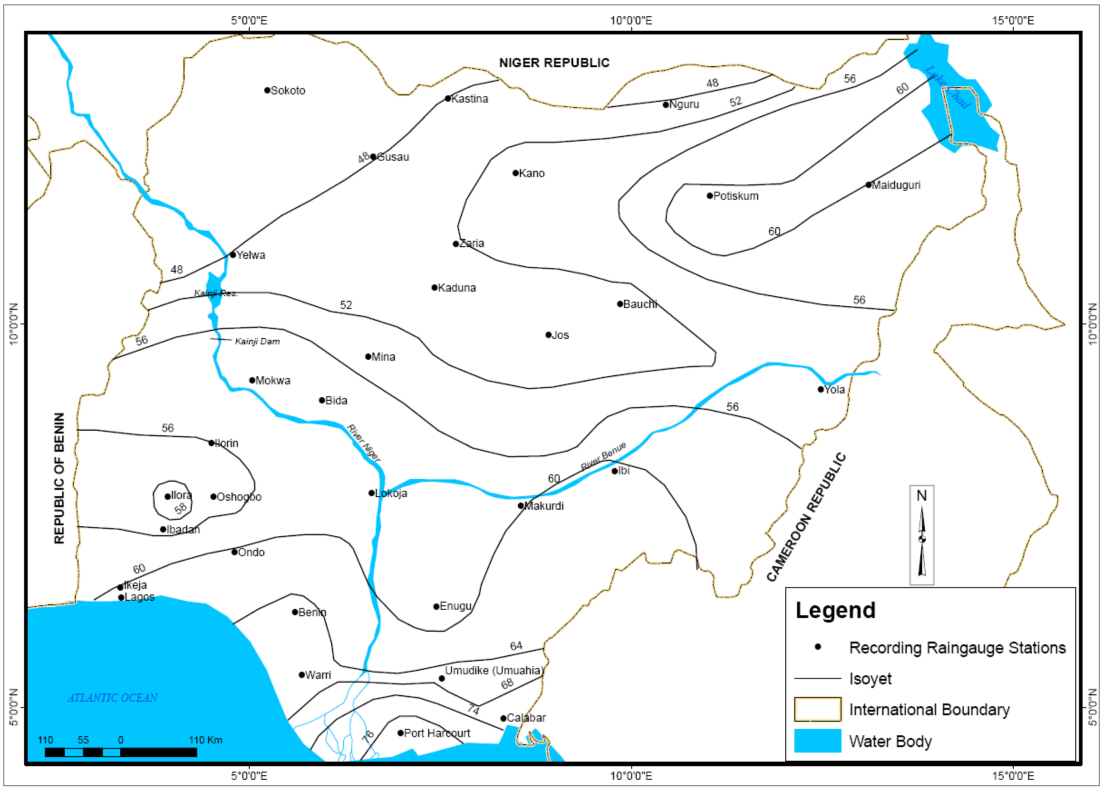


Figure 10.2: 10-year rainfall for duration of 30-minutes

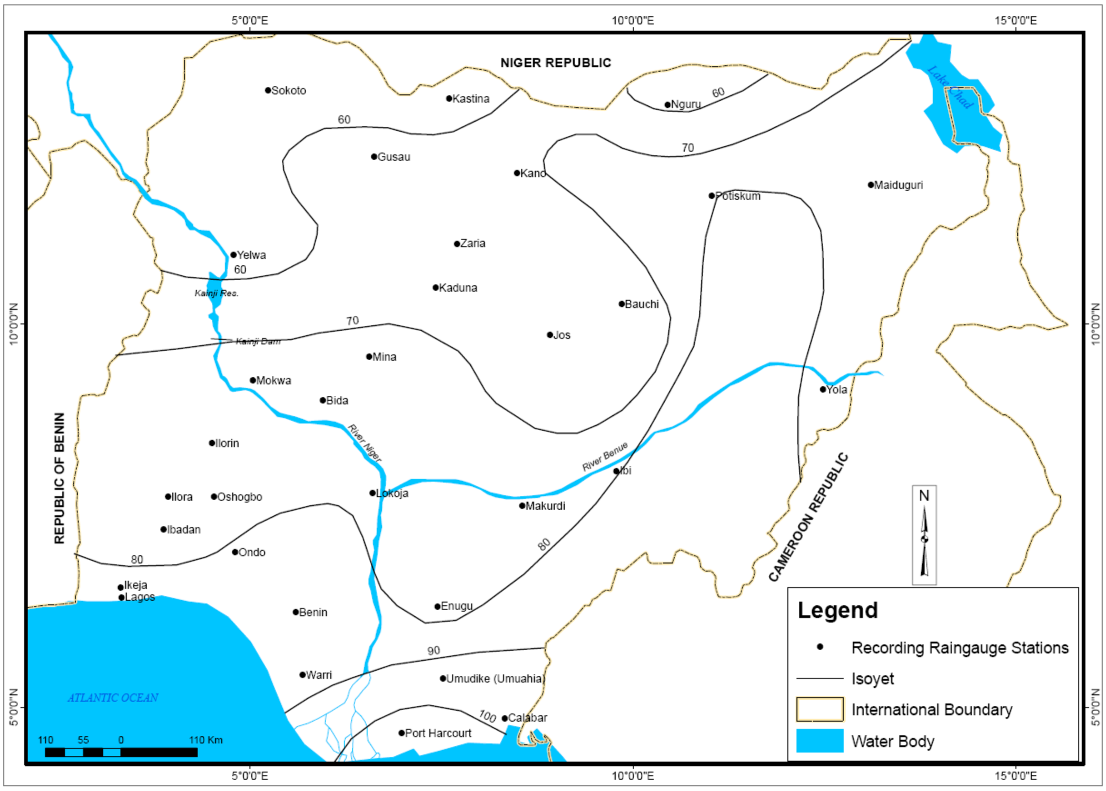


Figure 10.3: 10-year rainfall for duration of 1 hour

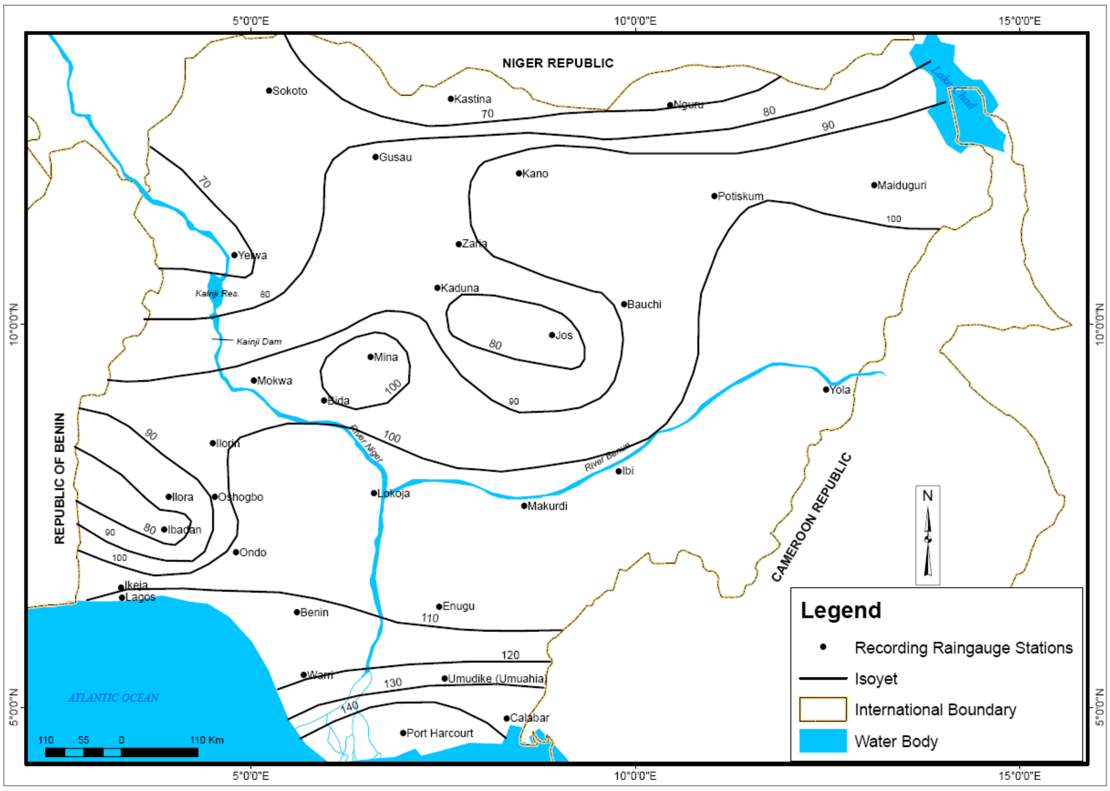


Figure 10.4: 10-year rainfall for duration of 3 hours

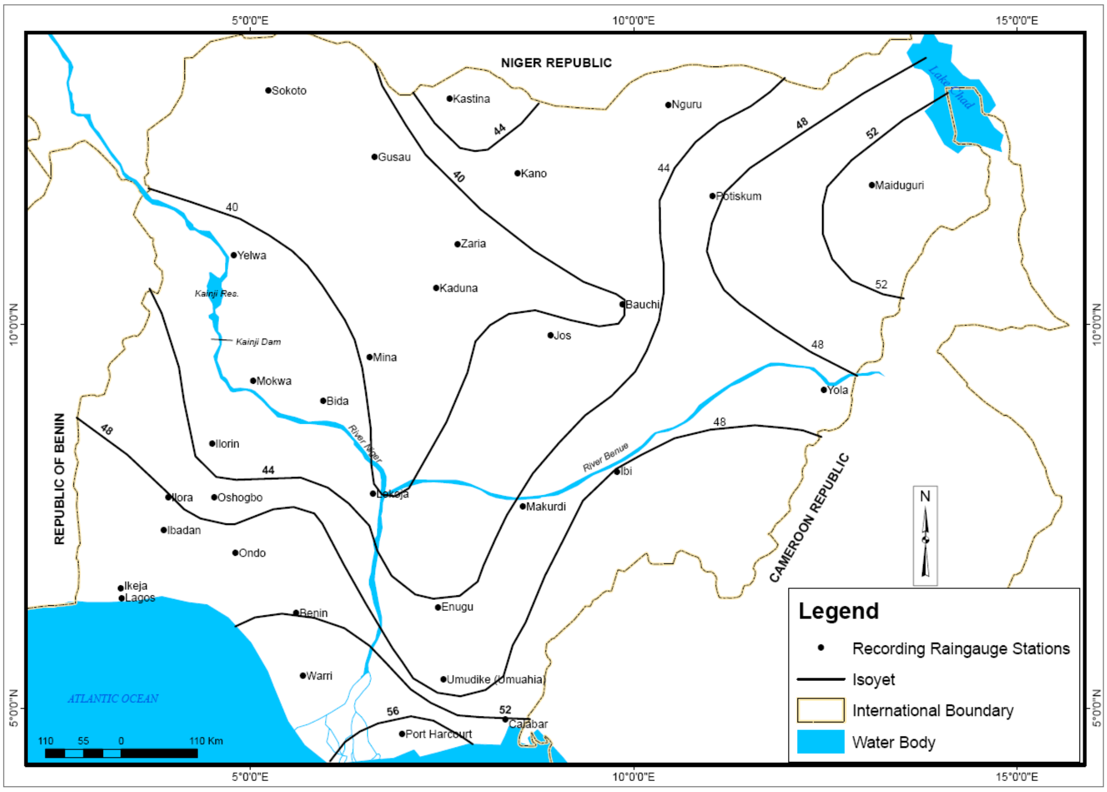


Figure 10.5: 25-year rainfall for duration of 15 minutes

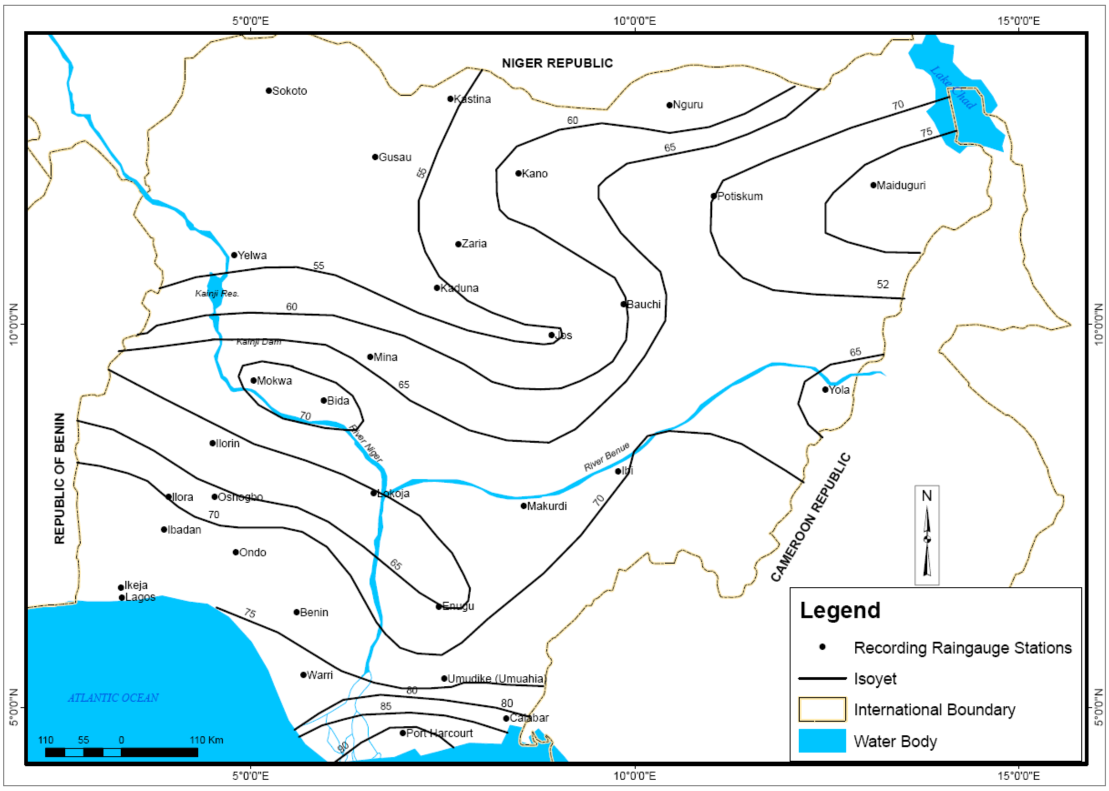


Figure 10.610.7: 25-year rainfall for duration of 30 minutes

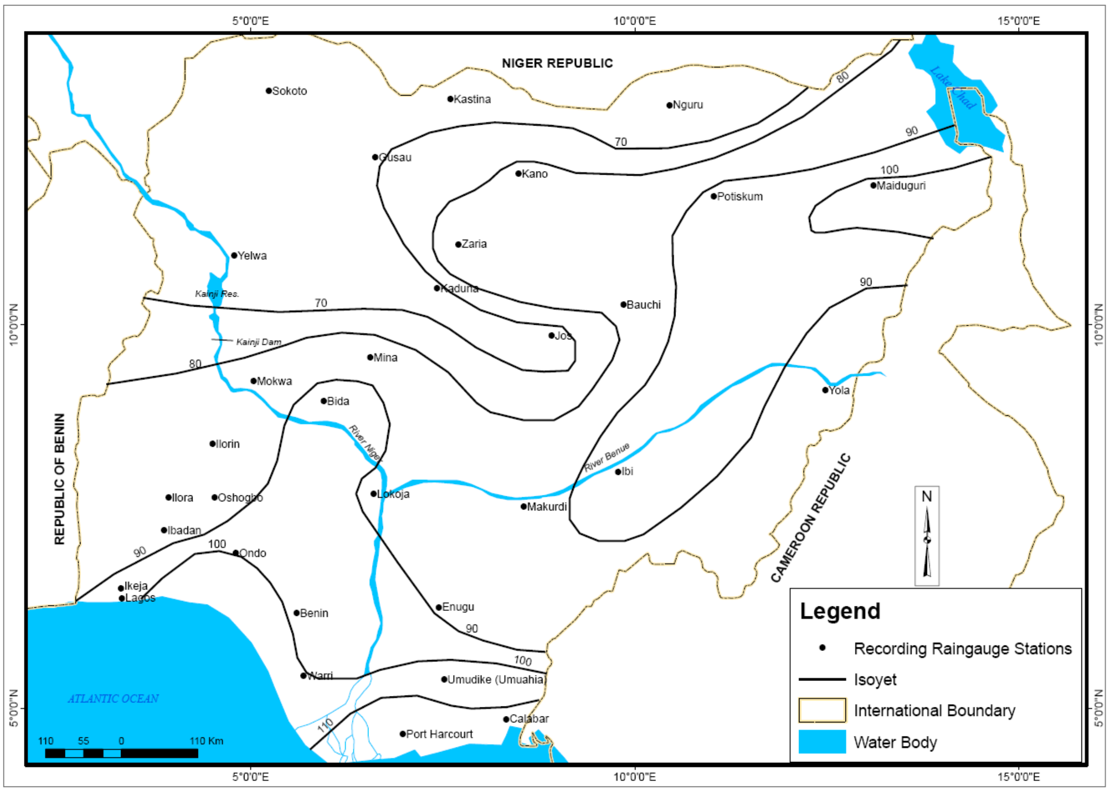


Figure 10.8: 25-year rainfall for duration of 1 hour

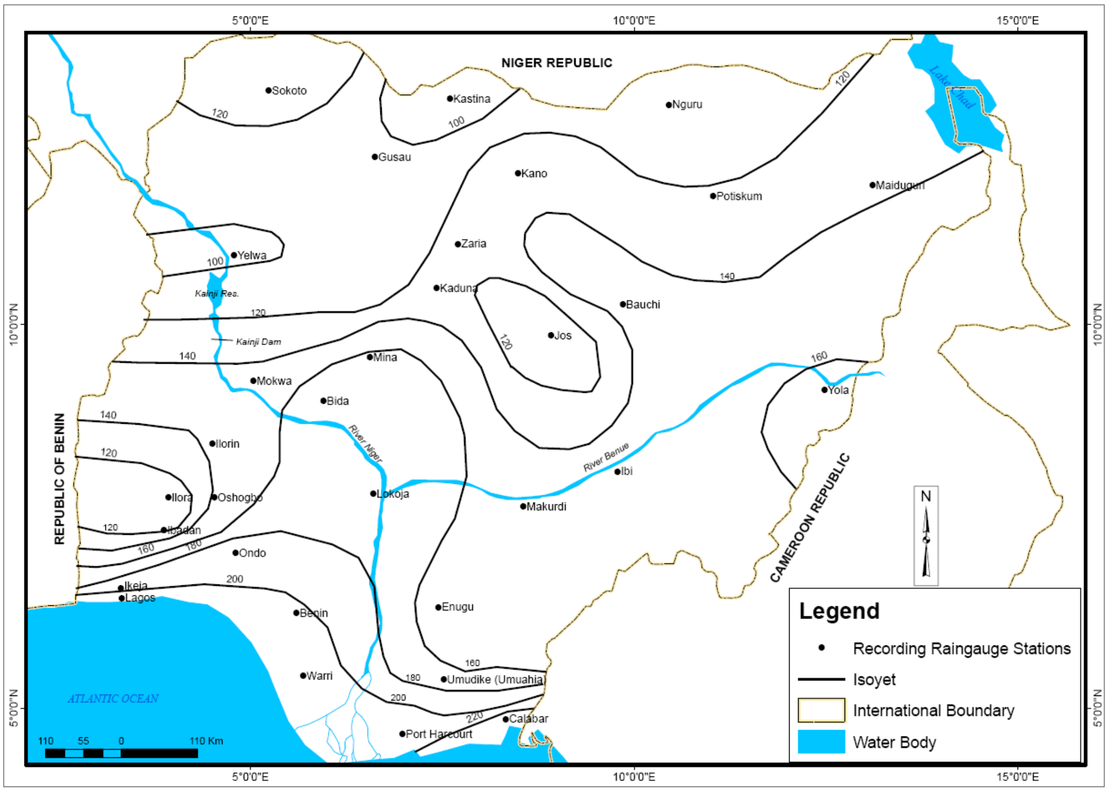


Figure 10.9: 25-year rainfall for duration of 24 hour

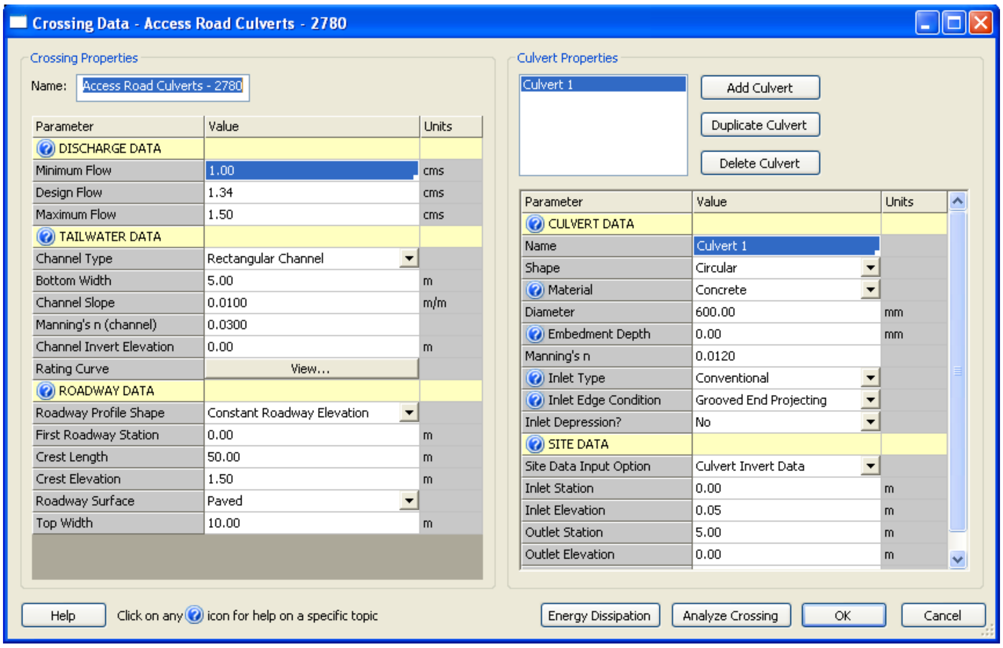
11: Appendix 2: Design Software

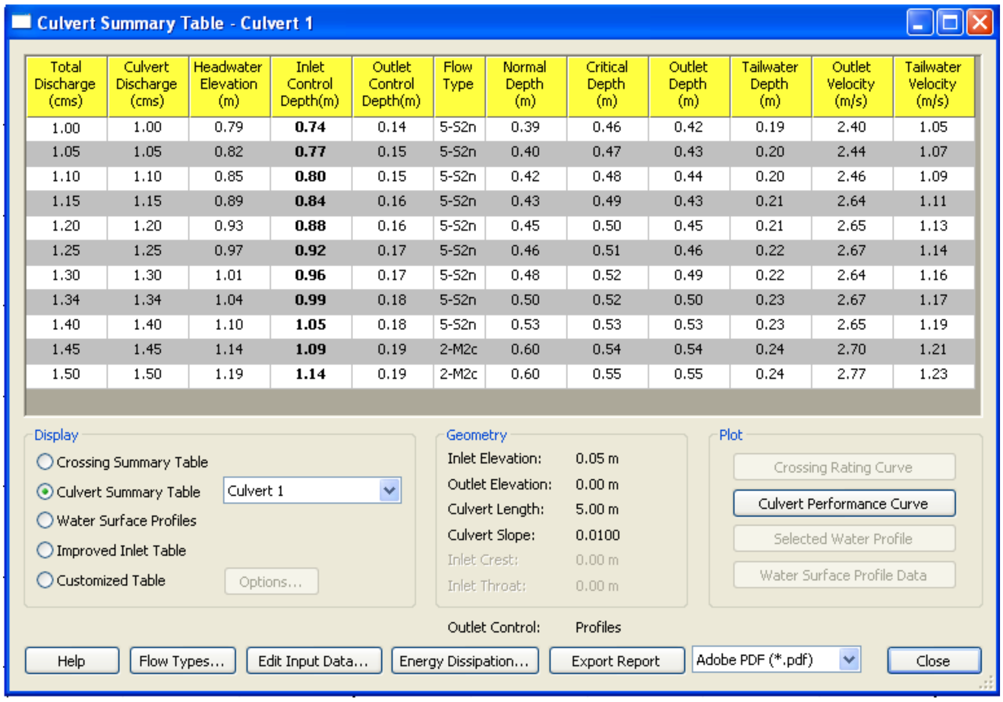
Several public domain hydraulic design software programs are available for free download from the internet and can be used to simplify and streamline the hydraulic design process. The following programs will be briefly described in this manual:

- HY-8
- HEC-RAS
- FHWA Hydraulic Toolbox
- EPA-SWMM

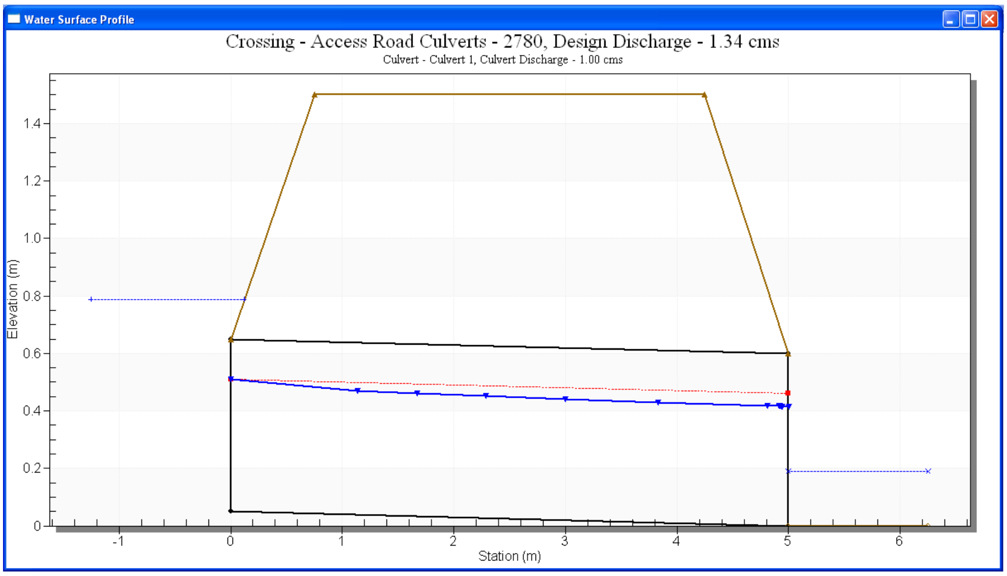
a. HY-8

HY-8, developed by the US Federal Highway Administration (FHWA), is a culvert design software program that can be freely downloaded from the FHWA website (www.fhwa.org) and is classified as public domain software. Input data are design discharge range, tailwater channel geometry, a roadway cross-section and an embankment template. Any commercially available culvert alternative material and size can easily be selected and a performance curve produced that is compared to design targets. The screen shots below show examples of the design input data screen, culvert results summary table screen and water surface profile screen for HY-8, respectively.





Total Discharge (cms)	Culvert Discharge (cms)	Headwater Elevation (m)	Inlet Control Depth(m)	Outlet Control Depth(m)	Flow Type	Normal Depth (m)	Critical Depth (m)	Outlet Depth (m)	Tailwater Depth (m)	Outlet Velocity (m/s)	Tailwater Velocity (m/s)
1.00	1.00	0.79	**0.74**	0.14	5-S2n	0.39	0.46	0.42	0.19	2.40	1.05
1.05	1.05	0.82	**0.77**	0.15	5-S2n	0.40	0.47	0.43	0.20	2.44	1.07
1.10	1.10	0.85	**0.80**	0.15	5-S2n	0.42	0.48	0.44	0.20	2.46	1.09
1.15	1.15	0.89	**0.84**	0.16	5-S2n	0.43	0.49	0.43	0.21	2.64	1.11
1.20	1.20	0.93	**0.88**	0.16	5-S2n	0.45	0.50	0.45	0.21	2.65	1.13
1.25	1.25	0.97	**0.92**	0.17	5-S2n	0.46	0.51	0.46	0.22	2.67	1.14
1.30	1.30	1.01	**0.96**	0.17	5-S2n	0.48	0.52	0.49	0.22	2.64	1.16
1.34	1.34	1.04	**0.99**	0.18	5-S2n	0.50	0.52	0.50	0.23	2.67	1.17
1.40	1.40	1.10	**1.05**	0.18	5-S2n	0.53	0.53	0.53	0.23	2.65	1.19
1.45	1.45	1.14	**1.09**	0.19	2-M2c	0.60	0.54	0.54	0.24	2.70	1.21
1.50	1.50	1.19	**1.14**	0.19	2-M2c	0.60	0.55	0.55	0.24	2.77	1.23



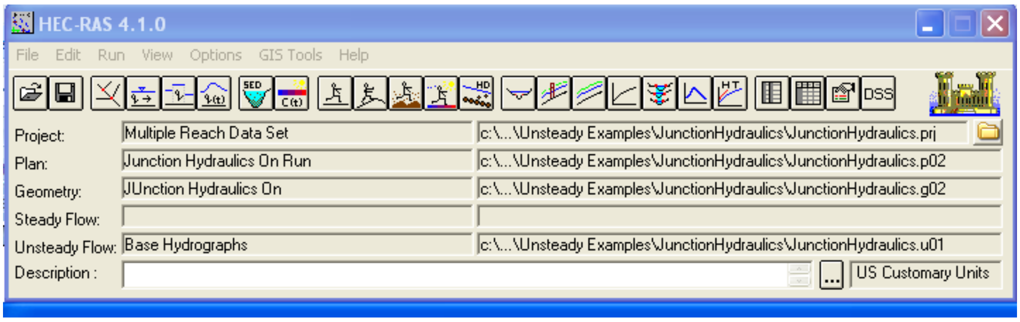
Further information on the theoretical basis of HY-8, its application and limitations can be found in the help documentation accompanying the program, as well as the FHWA publication, Hydraulic Design of Highway Culverts, 3rd Edition (2012).

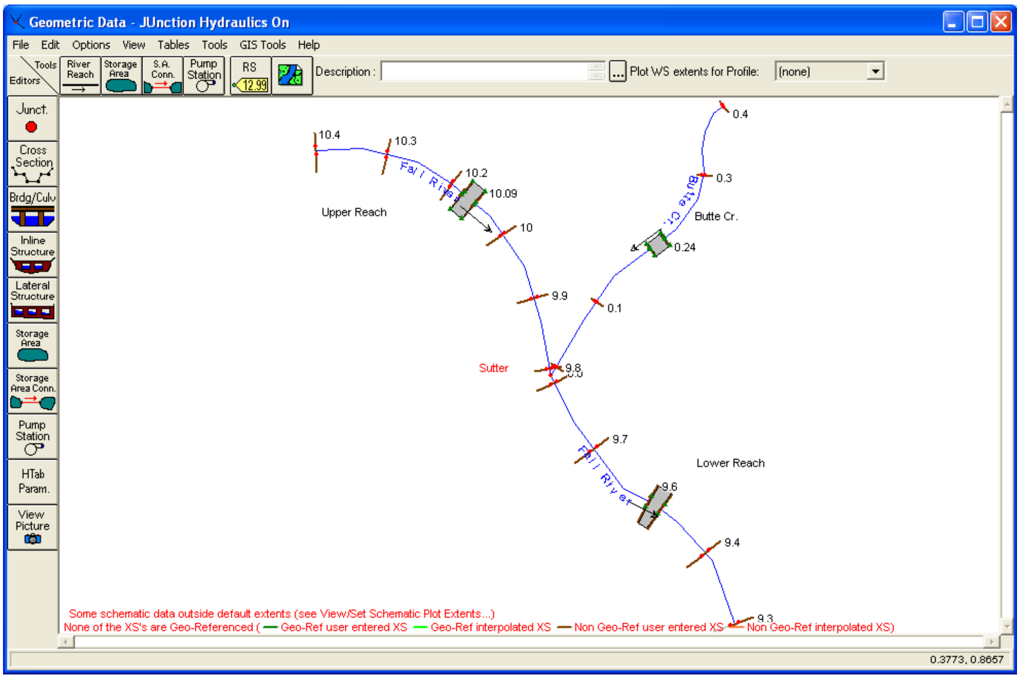
b. HEC-RAS

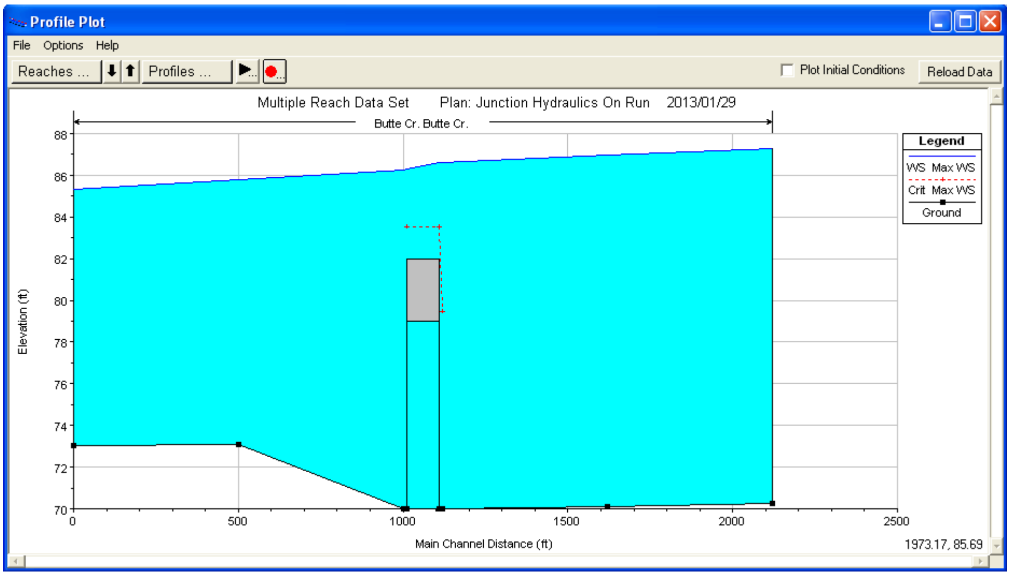
HEC-RAS is a public domain software program developed by the US Army Corps of Engineers for the calculation of water surface profiles in open channels. The program is one-dimensional, meaning that there is no direct modelling of the hydraulic effect of cross section shape changes, bends, and other two- and three-dimensional aspects of flow. The basic computational procedure of HEC-RAS for steady flow is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction and contraction / expansion. The momentum equation may be used in situations where the water surface profile is rapidly varied. These situations include hydraulic jumps, hydraulics of bridges, and evaluating profiles at river confluences.

For unsteady flow, HEC-RAS solves the full, dynamic, Saint-Venant equation using an implicit, finite difference method.

HEC-RAS is equipped to model a network of channels, a dendritic system or a single river reach. Certain simplifications must be made in order to model some complex flow situations using the HEC-RAS one-dimensional approach. It is capable of modelling subcritical, supercritical, and mixed flow regime flow along with the effects of bridges, culverts, weirs, and structures. Shown below are some examples of the HEC-RAS user interface and results output screens.







c. FHWA Hydraulic Toolbox

The FHWA Hydraulic Toolbox Program is a stand-alone suite of calculators that performs routine hydrologic and hydraulic computations. The program allows a user to perform and save hydraulic calculations in one project file, analyse multiple scenarios, and create plots and reports of these analyses. The program is public domain software and can be freely downloaded from the FHWA website (www.fhwa.dot.gov).

Ten calculators are available for project development:

(i) *Channel Analysis* – allows the user to solve for either flow or depth for a range of channel shapes, including trapezoidal, triangular, circular and irregular (user defined).

(ii) *Channel Lining Design Analysis* – allows the user to design stable side drains/ channels using four available channel lining types based on allowable shear stresses: rock (small riprap, cobble, and gravel), vegetation (grass, etc.), rolled erosion control product (RECP) and gabion mattress.

(iii) *Weir Analysis* – offers seven types of weir for analysis, including rectangular, trapezoidal and v-notch (with various angles) and allows the user to solve for either head or flow.

(iv) *Kerb and Gutter Analysis* – allows the user to calculate the design flow or the width of spread for uniform or compound gutters.

(v) *Median/Ditch Drop-Inlet Analysis* – computes the amount of flow captured and by-passed by a typical drop-inlet placed in the bottom of median, roadside, or similar ditch/ drain.

(vi) *Rational Method Hydrologic Analysis* – can be used to enter the variables required to compute discharge rate by the Rational Method. The calculator is setup for North American conditions, but does allow for entry of user specified IDF data for other locations.

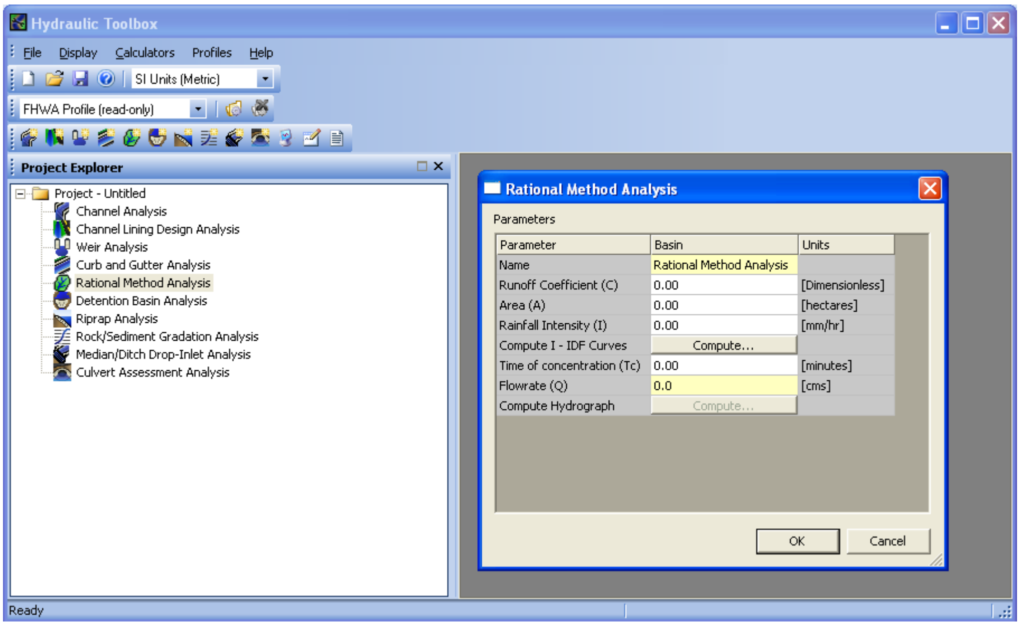
(vii) *Detention Basin Analysis* – allows the user to route an inflow hydrograph through a detention basin to generate an outflow hydrograph. The calculator allows the user to specific the inflow hydrograph, the storage relationship for the basin and the outlet structure dimensions and characteristics.

(viii) *Riprap Analysis* – offers eight applications for designing rock riprap armour protection / scour countermeasures, including channel revetment, bridge piers, bridge abutments, embankment overtopping, culvert outlets and wave attack.

(ix) *Rock—Sediment Gradation Analysis* – allows the user to compute gradation information from pebbles counts conducted in the field or from high resolution digital images (photographs) of any surface composed of discrete particles.

(x) *Culvert Assessment Analysis* – a project-level tool that identifies suggested culvert rehabilitation, repair, and replacement methods based on the findings of field assessments conducted in accordance with FHWA’s Culvert Assessment and Decision-Making Procedures.

There are modules that save notes and reports with the analysis results. These results can be printed at the user's discretion.



d. EPA-SWMM

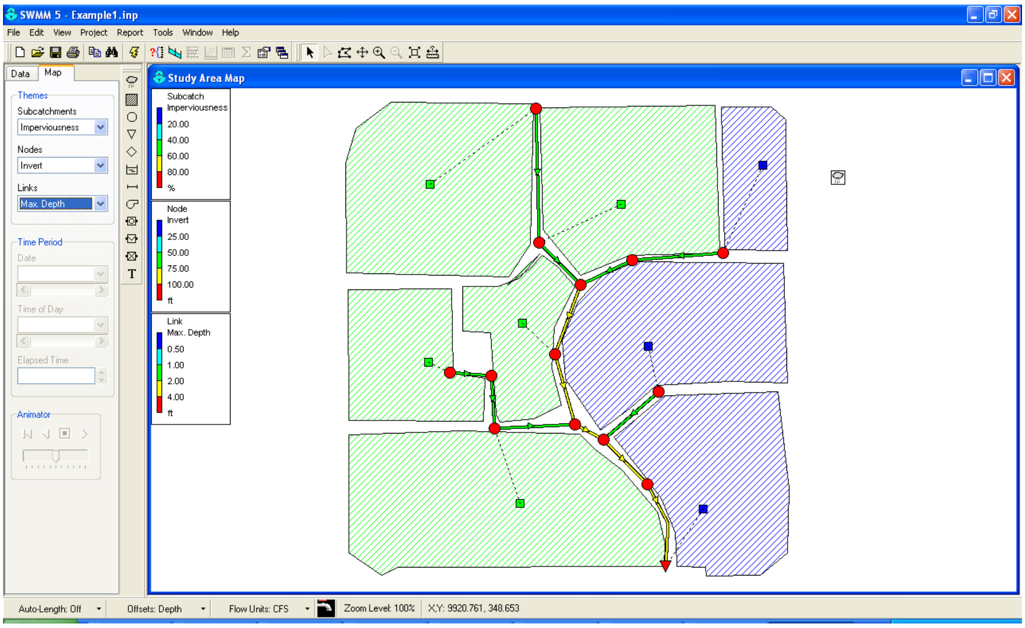
The United States Environmental Protection Agency's Stormwater Management Model (SWMM) was first developed in 1971, and has since undergone several major upgrades. It continues to be widely used throughout the world for planning, analysis and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban areas as well.

This general-purpose urban hydrology and conveyance system hydraulics software is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub-catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps.

EPA has recently extended SWMM 5 to explicitly model the hydrologic performance of specific types of low impact development (LID) controls, such as porous pavement, bio-retention areas (e.g., rain gardens, green roofs, and street planters), rain barrels, infiltration trenches, and vegetative swales. The updated model allows engineers and planners to accurately represent any combination of LID controls within a study area to determine their effectiveness in managing stormwater and combined sewer overflows.

Running under Windows, SWMM 5 provides an integrated environment for editing study area input data, running hydrologic, hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded drainage area and conveyance system maps, time series graphs and tables, profile plots, and statistical frequency analyses.

The screen shot below shows an example of the main user interface for EPA-SWMM 5.



12: Appendix 3: Culvert Design Nomographs

INLET-CONTROL NOMOGRAPHS

Figure 12.1 through Figure 12.6

Instructions for Use

a. *To determine headwater (HW)*

Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or Q/B for box culverts; mark intersection of straightedge or HW/3; scale marked. (1).

If HW/D scale marked (1) represents entrance type used, read HW/D on scale (1). If some other entrance type is used extend the point of intersection in (a) horizontally on scale (2) or (3) and read HW/D.

Compute HW by multiplying HW/D by D.

b. *To determine culvert size*

Given an HW/D value, locate HW/D on scale for appropriate entrance type. If scale (2) or (3) is used extend HW/D point horizontally to scale (1).

Connect point on HW/D scale (1) as found in (a) above to given discharge and read diameter, height of size of culvert required.

c. *To determine discharge (Q)*

Given HW and D, located HW/D on scale for appropriate entrance type. Continue as in 2(a).

Connect point on HW/D scale (1) as found in (a) above and the size of culvert on the left scale and read Q or Q/B on the discharge scale.

(C) If Q/B is read in (b) multiply by B to find Q.

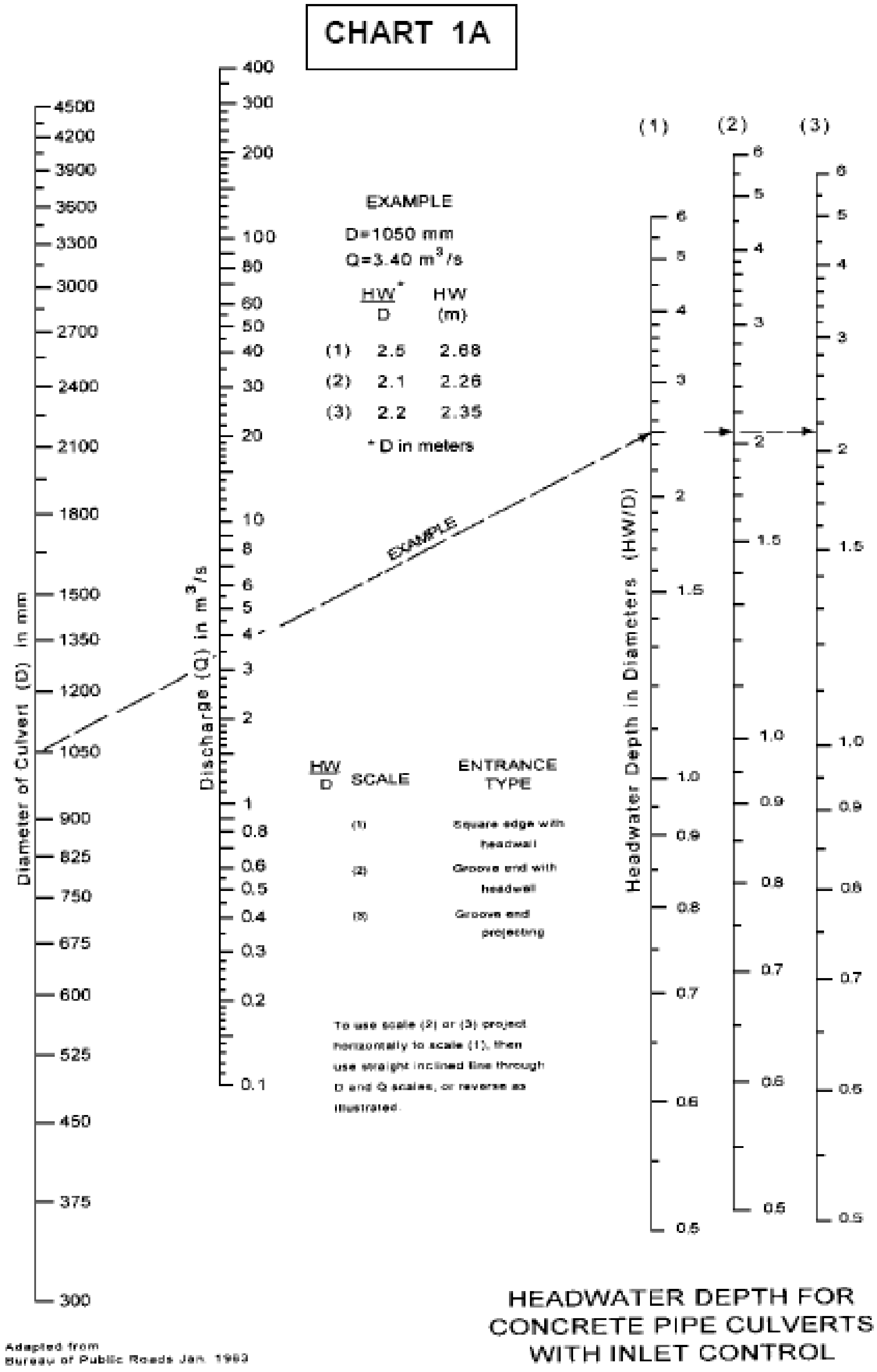


Figure 12.1: Inlet Control Nomograph for Concrete Pipe Culverts

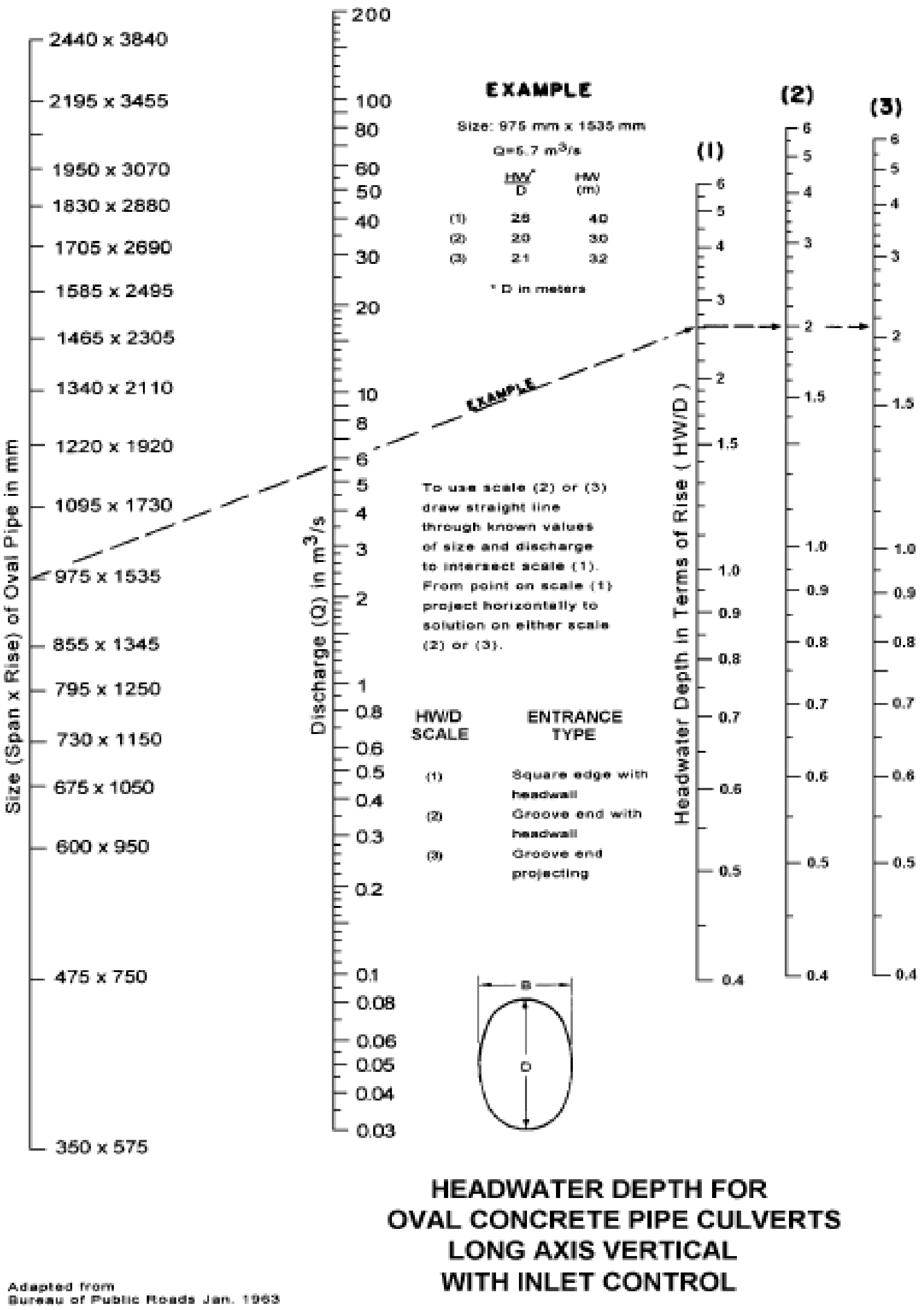


Figure 12.2: Inlet Control Nomograph for Oval Concrete Pipe Culverts-Long Axis Vertical

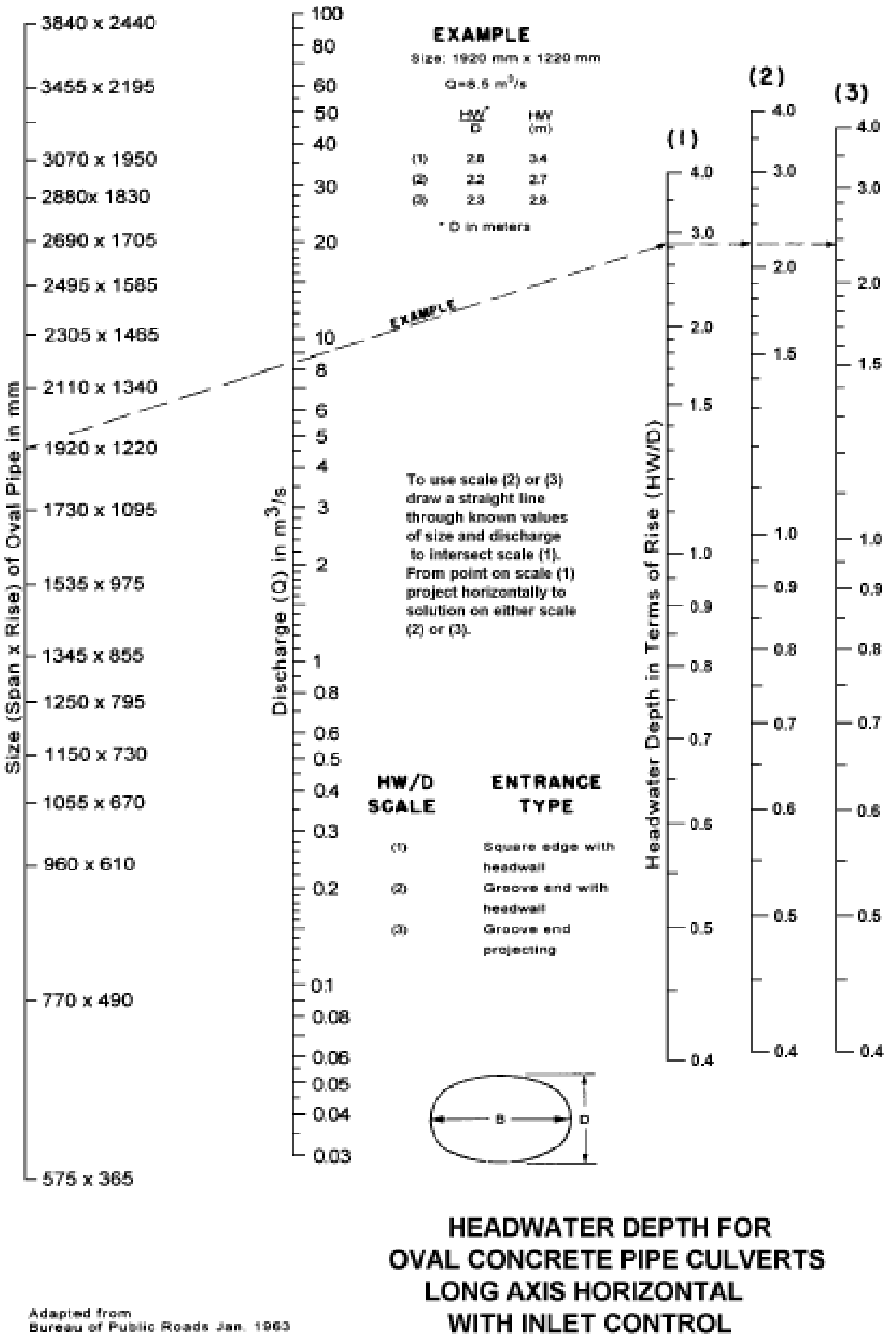


Figure 12.3: Inlet Control Nomograph for Oval Concrete Pipe Culverts-Long Axis Horizontal

Figure 12.4: Inlet Control Nomograph for C.M. Pipe Culverts

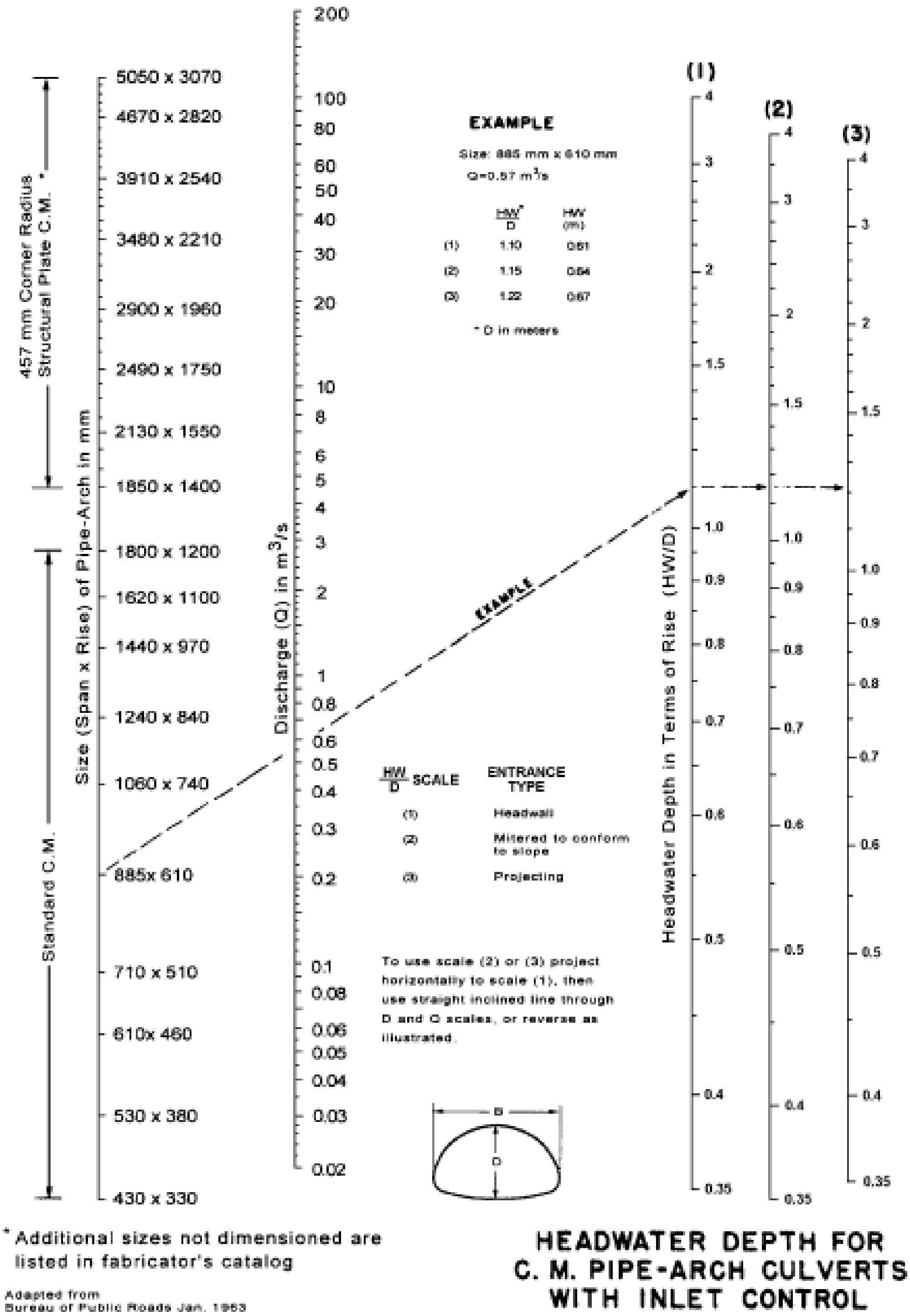


Figure 12.5: Inlet Control Nomograph for C.M. Pipe-Arch Culverts

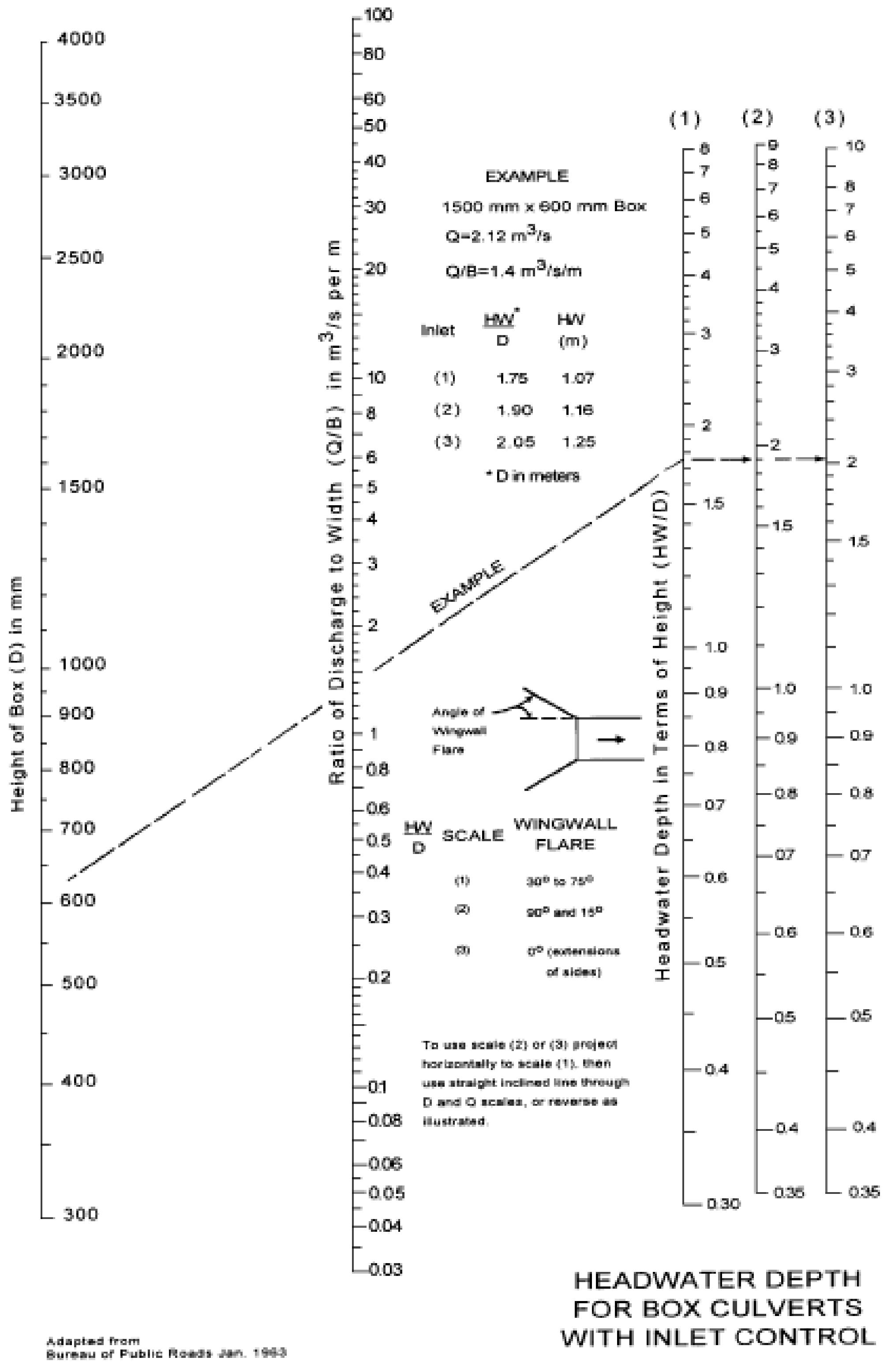


Figure 12.6: Inlet Control Nomograph for Box Culverts

OUTLET-CONTROL NOMOGRAPHS

Figure 12.7 through Figure 12.20

Instructions for Use

These nomographs solve Equation 5 – 5: for head H when culverts flow full with outlet control. They are also used in approximating the head for some part full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW.

- To determine head H for a given culvert and discharge, Q
    - Locate appropriate Nomographs for type of culvert selected. .

- Begin nomographs solution by locating starting point on the length scale.
    - To locate the proper starting point on the length scales follow instructions below.

a. If the “n” value of the nomographs corresponds to that of the culvert being used, find the proper K_e from Table 1-307 (VII) and on the appropriate nomographs locate starting point on length curve for that K_e. If a K_e curve is not shown for the selected K_e see (b) below. If “n” value for the culvert selected differs from that of the Nomographs, see (c) below.

b. For the “n” of the Nomographs and a K_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the K_e values.

c. For a different value of roughness coefficient n_1 than that of the chart "n", use the length scales shown with an adjusted length L_1, calculated by the formula $L1 = L (n_1/n^2)$. See instruction for "n" values.

d. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line". See instruction 3 for size considerations for rectangular box culvert.

e. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head (in rnetres) on the head (H)- scale. For values beyond the limit of the chart scales, find H by solving equation given on nomographs or by $H = KQ^2$ where K is found by substituting values of H and Q from chart.

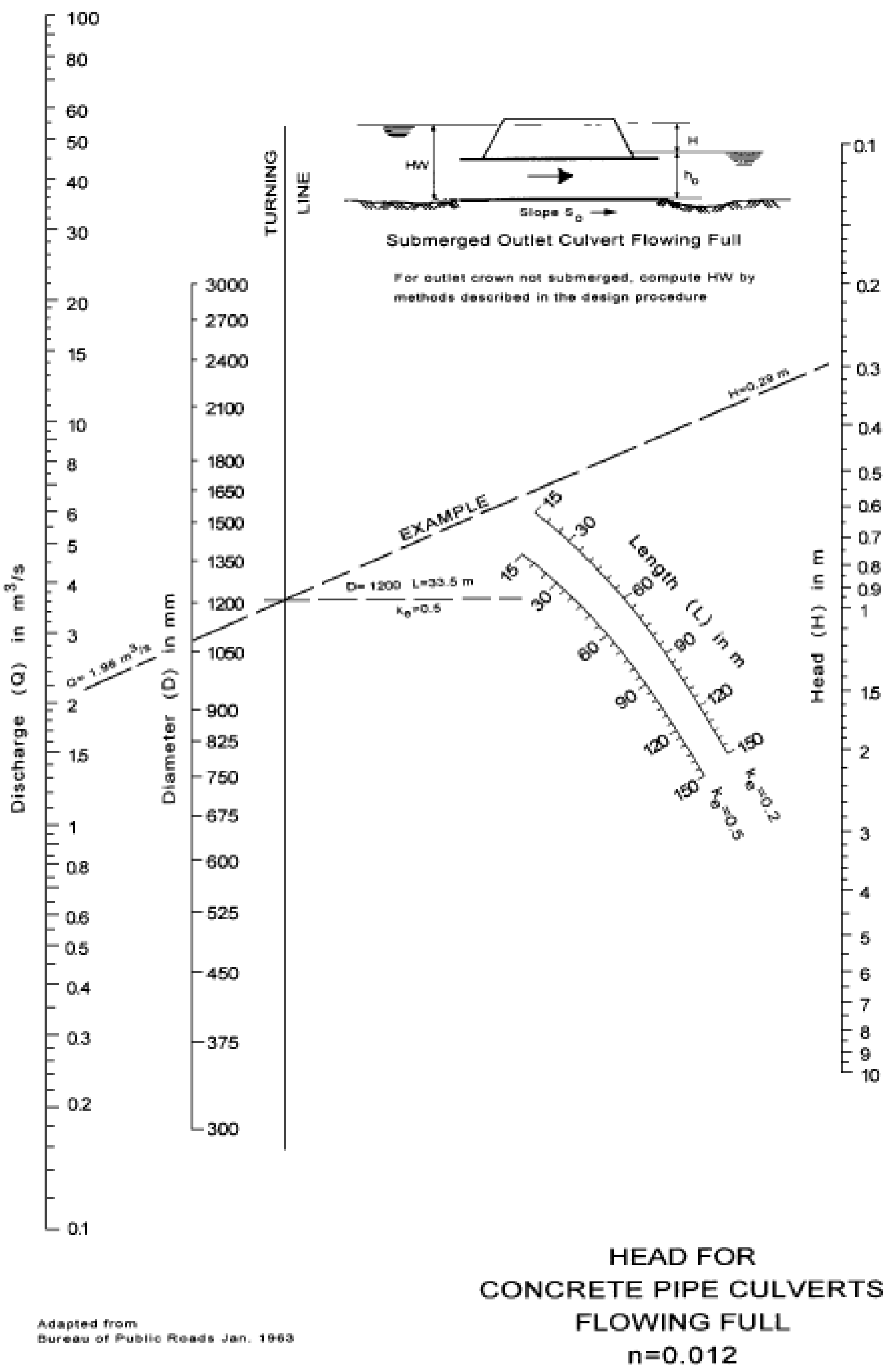
- Find the "n" value for the culvert selected by using the table below: -
  - Concrete Pipe
  - Vitrified Clay Pipe

Smooth-Flow C.M.C.P	n = 0.012
C.M.C.P. Asphalt Coated and 40% Paved Invert	n = 0.019
Plain Metal Culvert Pipe and Asphalt Coated	n = 0.024
Structural Plate Pipe & Plate Pipe Arches	n = 0.0302 to 0.0328
75mm x 25mm Corrugations Plain	n = 0.027
75mm x 25mm Corrugations 40% Paved Invert	n = 0.021

- To use the box culvert nomograph. Figure 1-307.24 for full flow for other than square boxes.
  - Compute cross-sectional area of the rectangular box. Note 3/

- Connect proper point (see instruction 1) on t length scale to barrel area (Note 3/) and mark point on turning line.
- Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in metre on the head (H) scale.

Note 3/ The area scale on the nomographs is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 2/3D. For other box proportions use equation shown on nomograph for more accurate results.



With long axis vertical or horizontal flowing full with n=0.012

Figure 12.7: Outlet Control Nomograph for Pipe Culverts flowing full with n=0.012

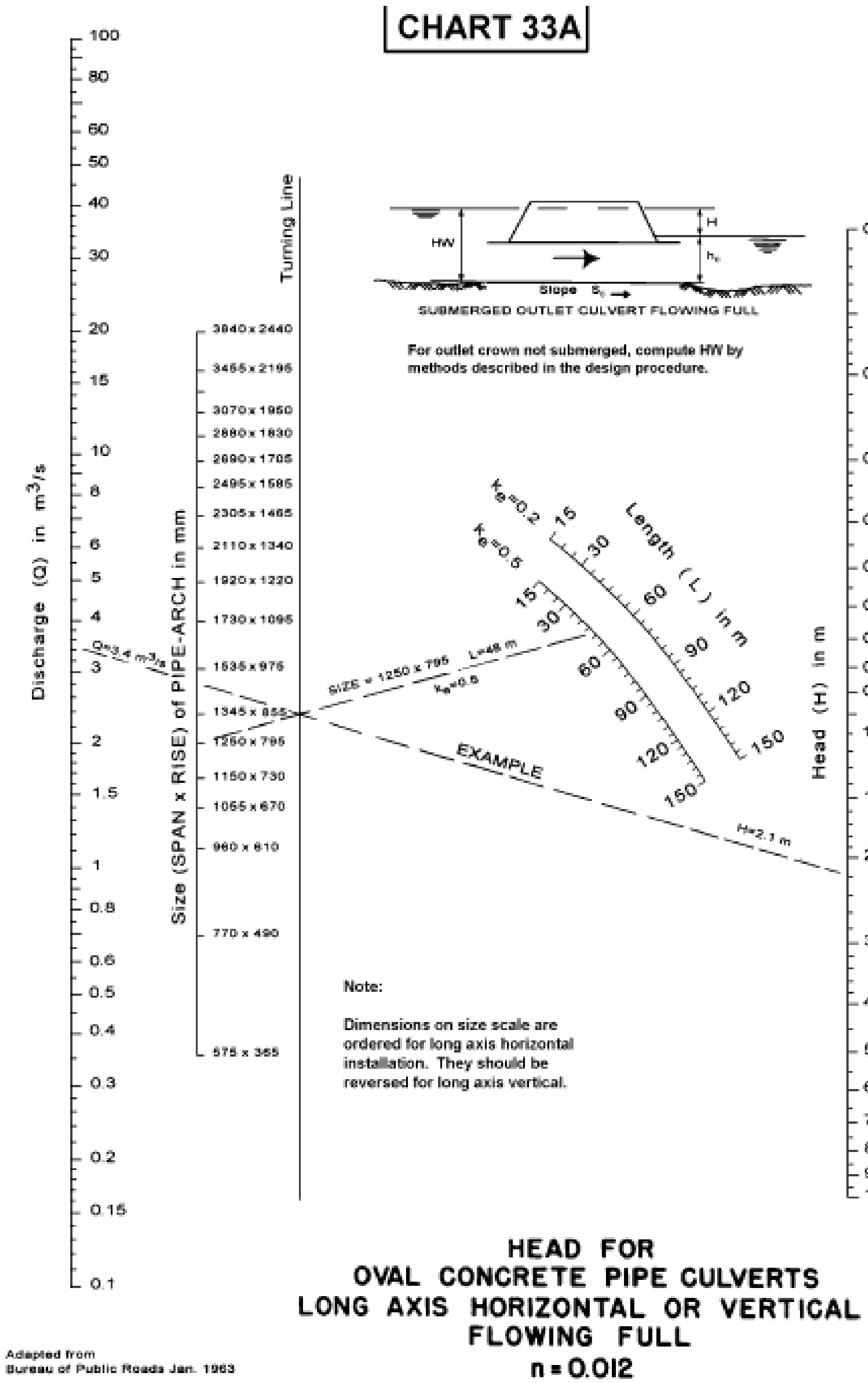


Figure 12.8: Outlet Control Nomograph for Oval Concrete Pipe Culverts

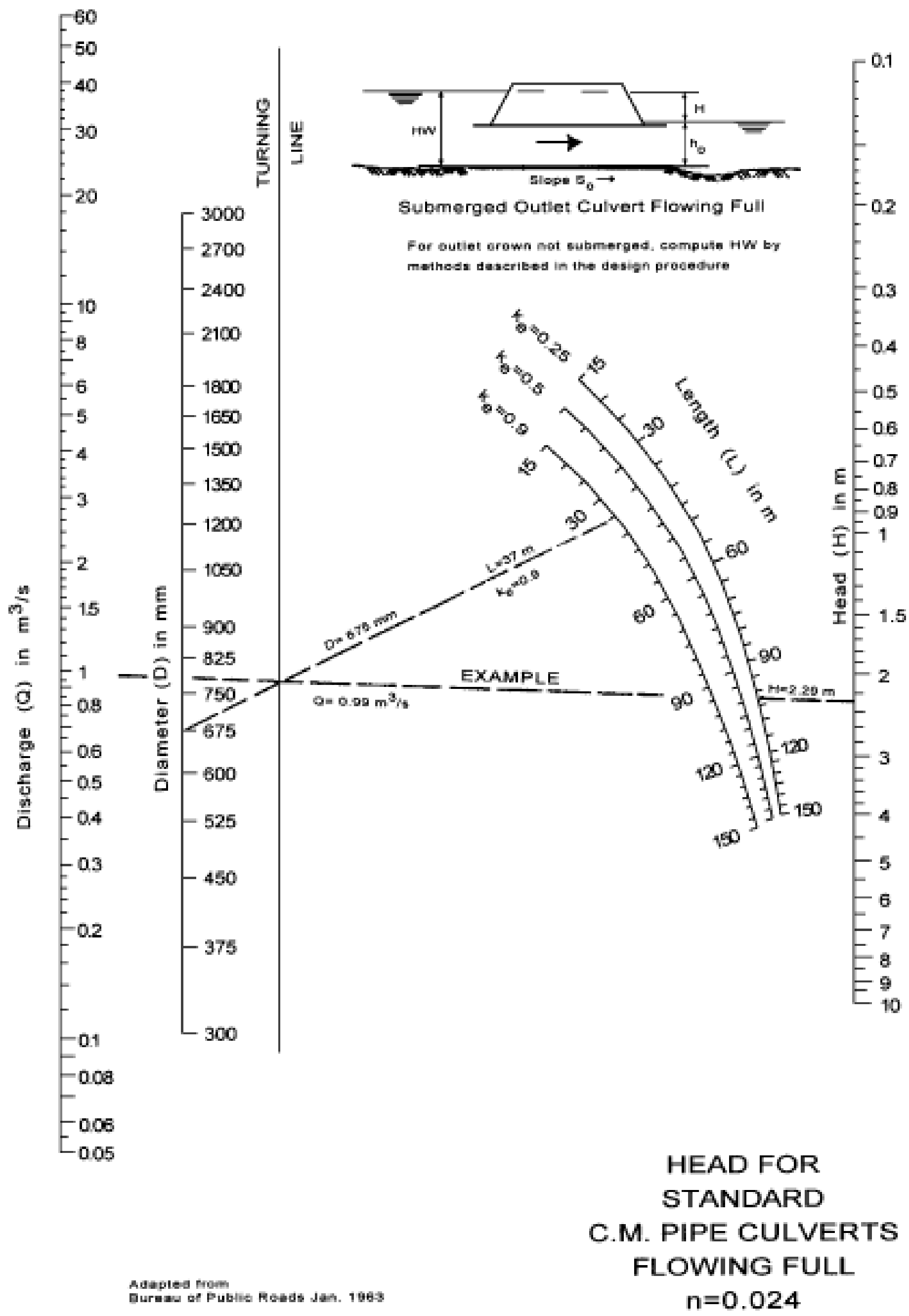


Figure 12.9: Outlet Control Nomograph for Standard C.M. Pipes

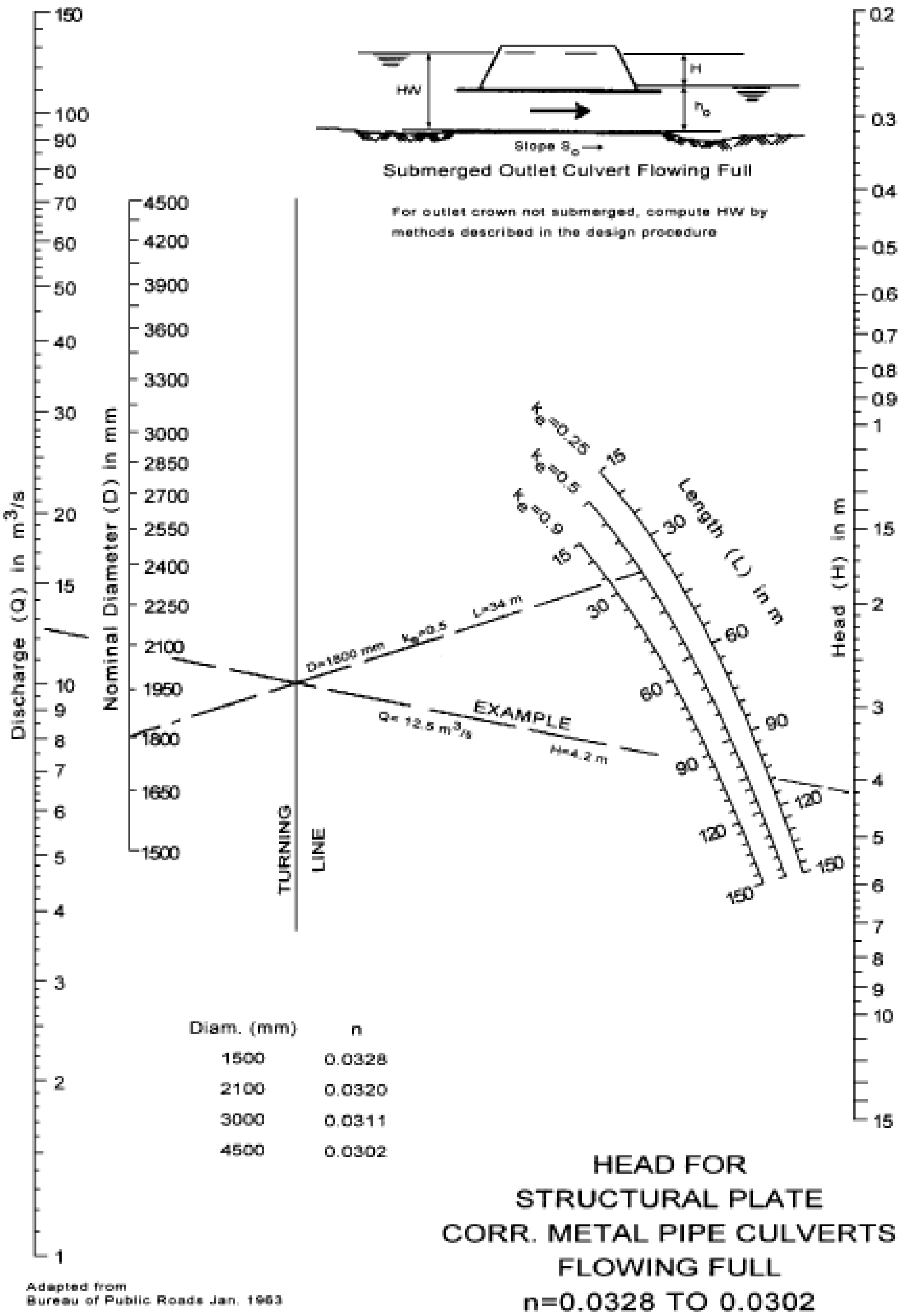


Figure 12.10: Outlet Control Nomograph: Corrugated Metal Pipe Culverts Flowing Full

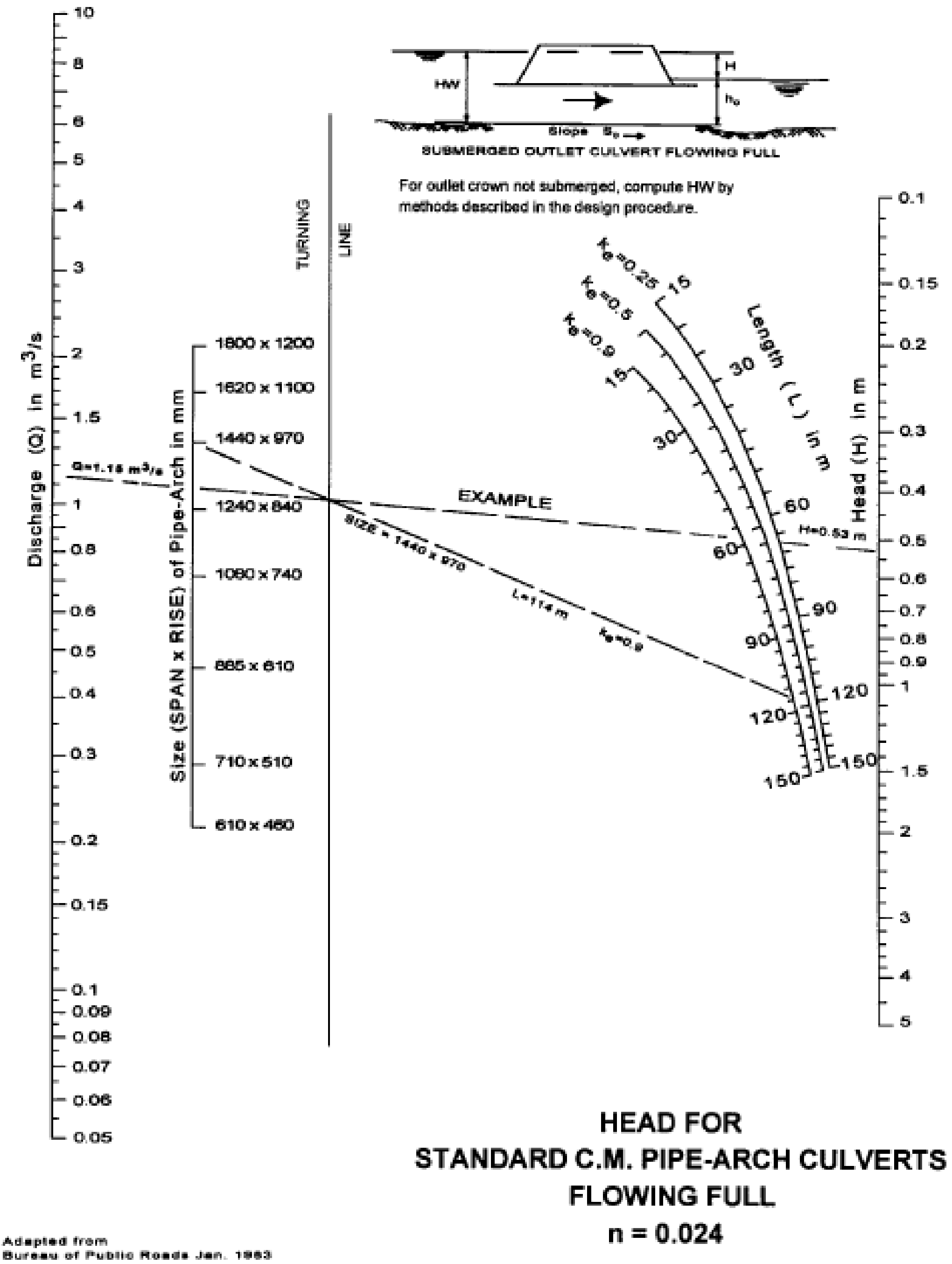


Figure 12.11: Outlet Control Nomograph for C.M. Pipe-Arch Culverts flowing full

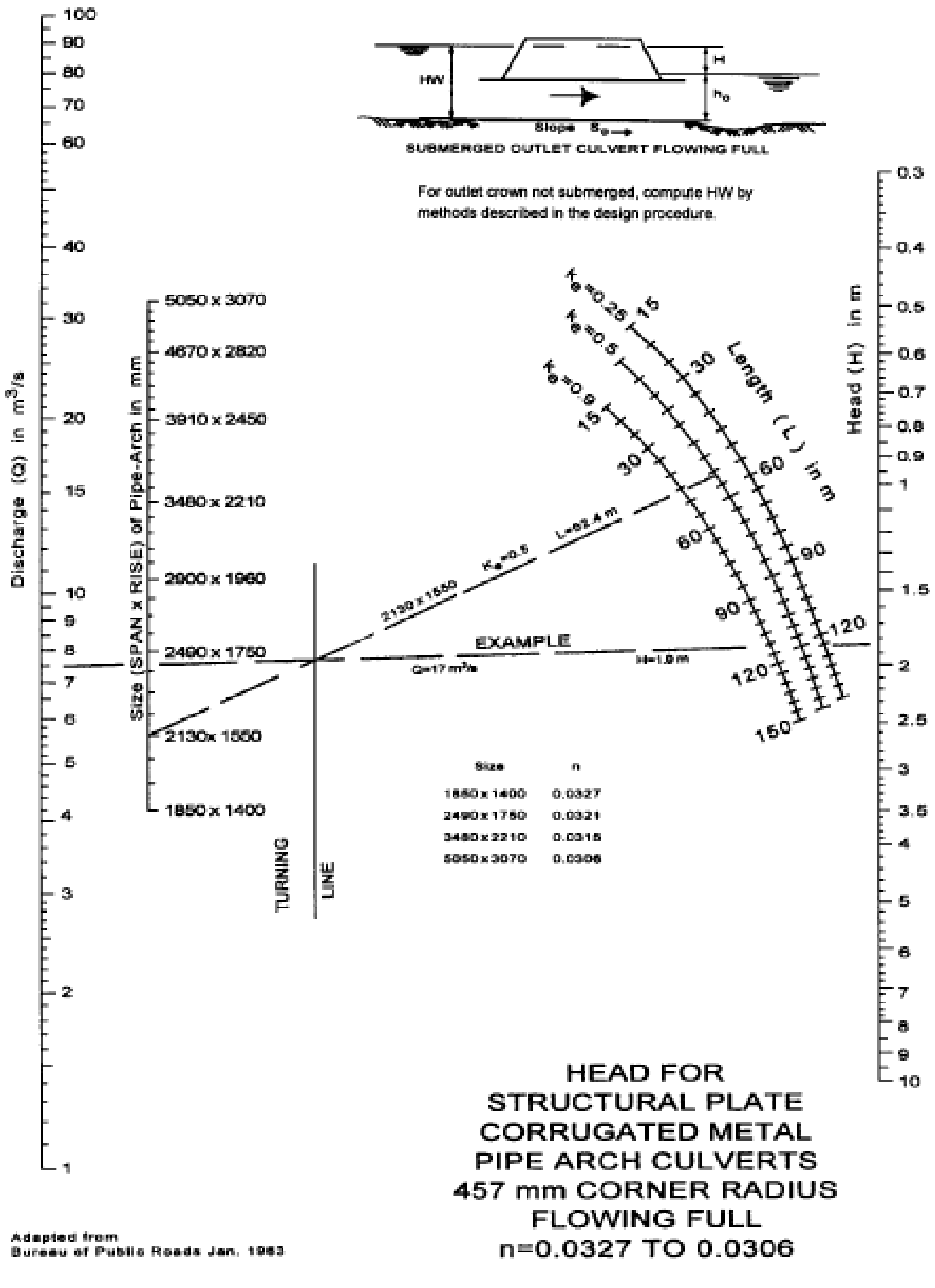


Figure 12.12: Outlet Control Nomograph: Corrugated Metal pipe arch culverts

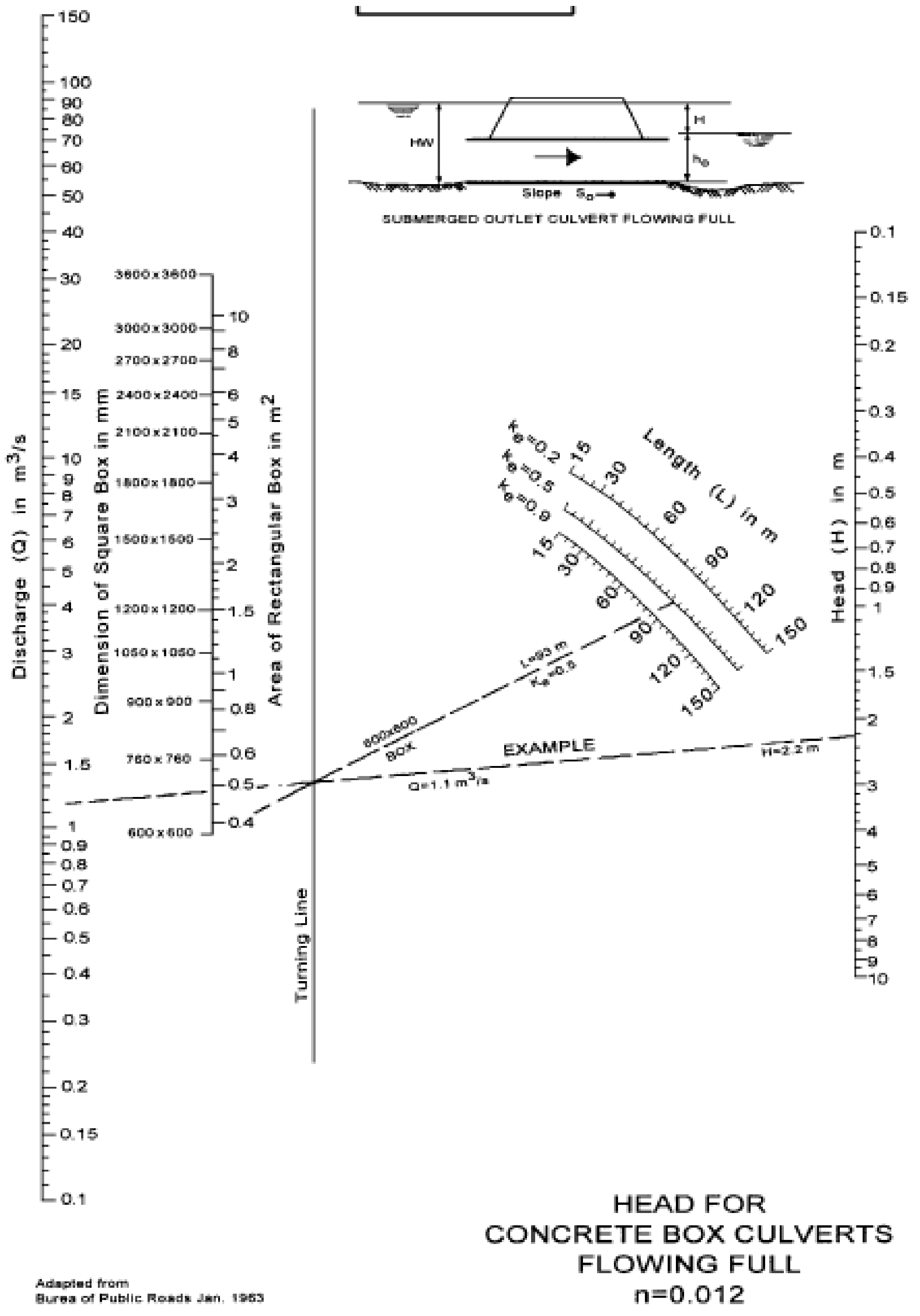


Figure 12.13: Outlet Control Nomograph: Concrete Box Culverts flowing full

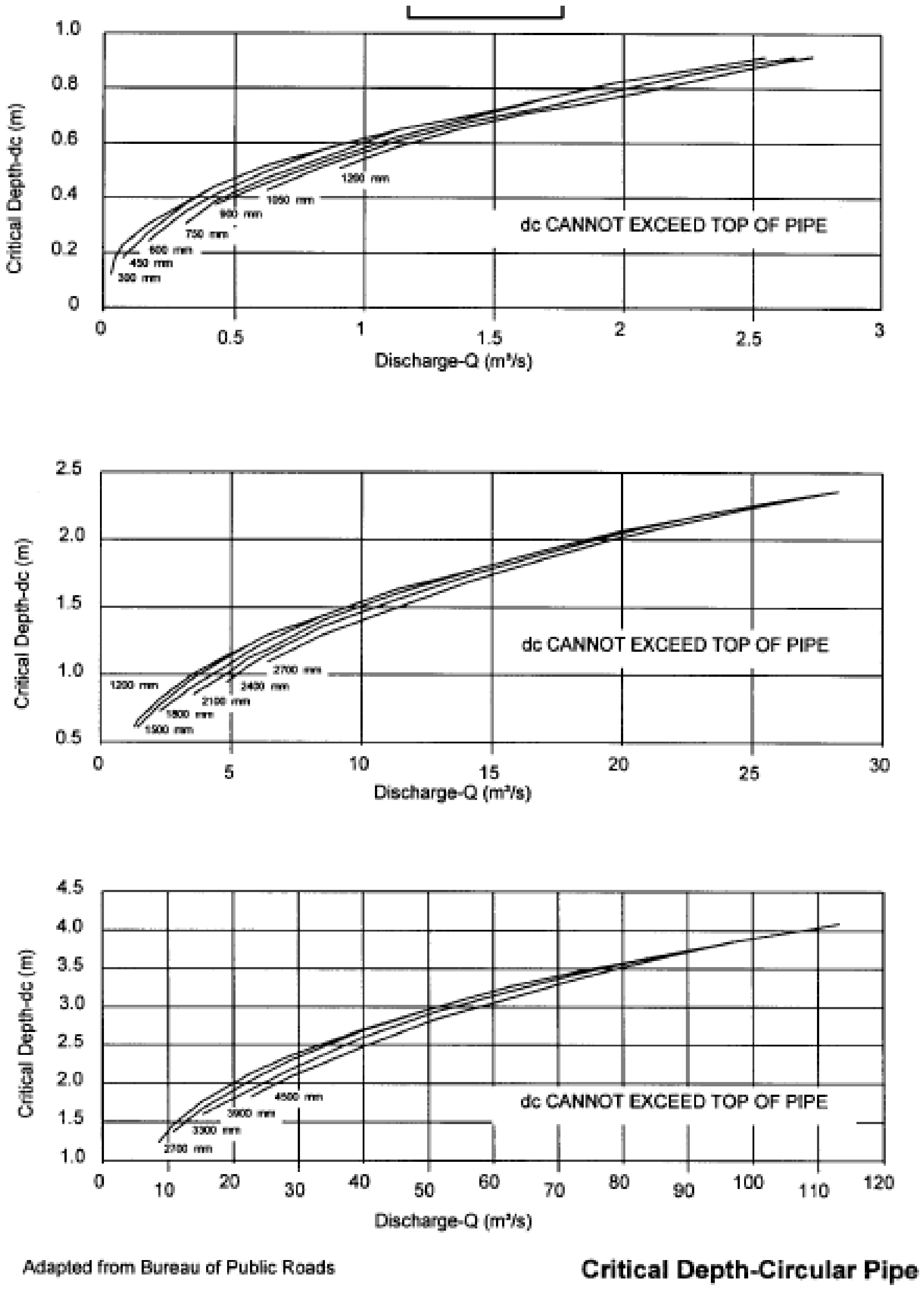


Figure 12.14: Outlet Control Nomograph: Critical Depth-Circular Pipe

Long Axis Horizontal

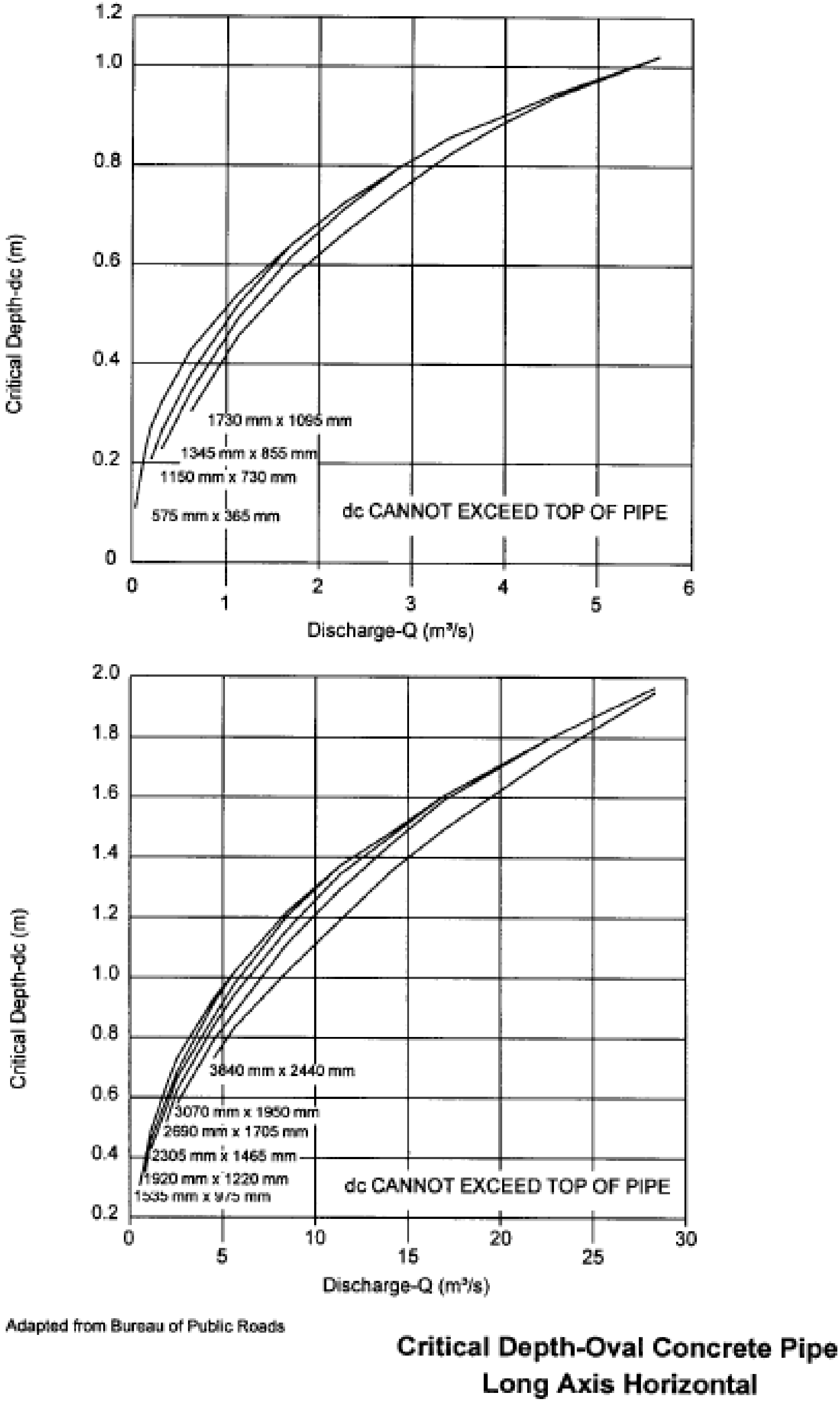


Figure 12.15: Outlet Control Nomograph: Critical Depth - Oval Concrete Pipe H

Long Axis Vertical

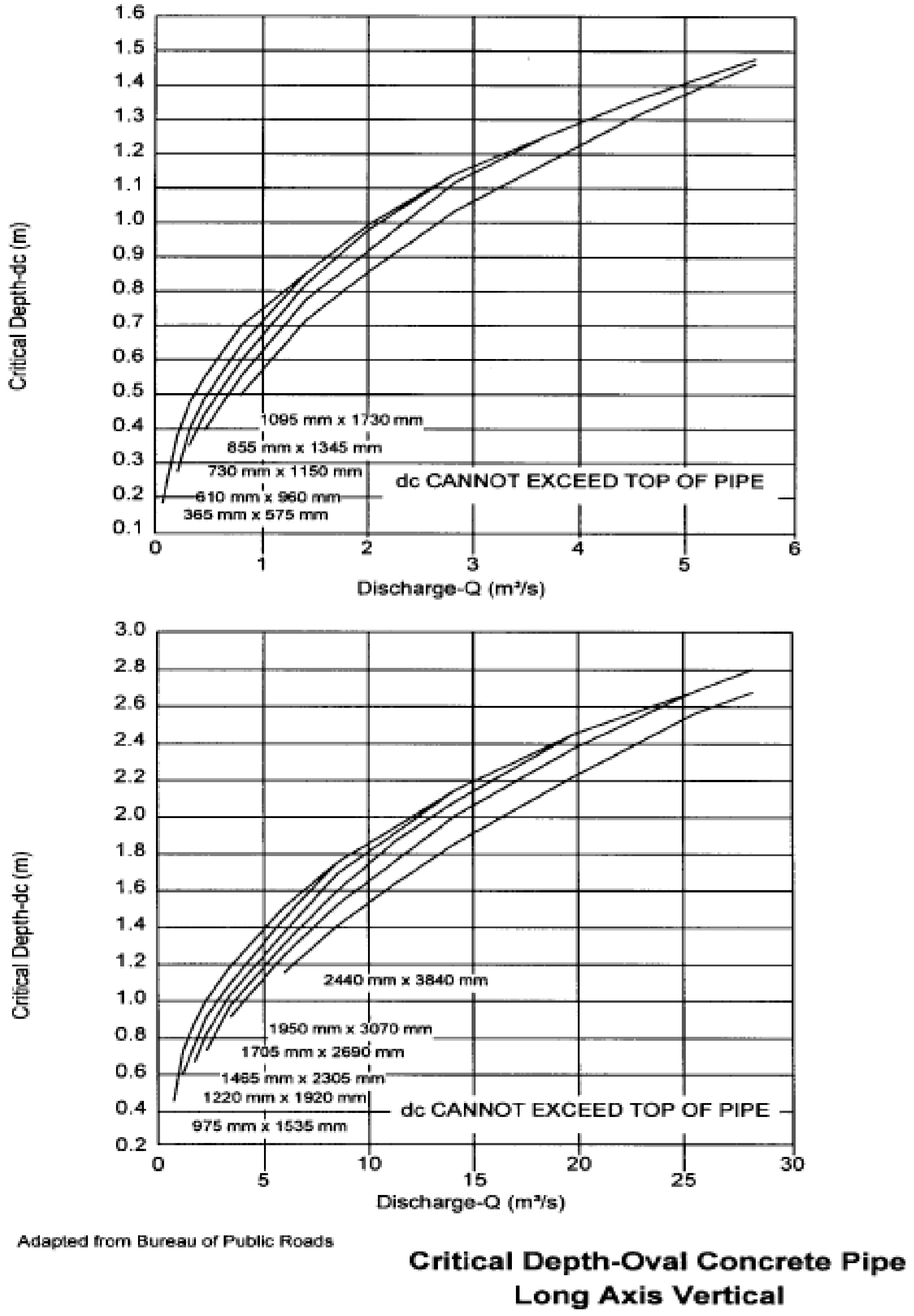


Figure 12.16: Outlet Control Nomograph: Critical Depth - Oval Concrete Pipe V

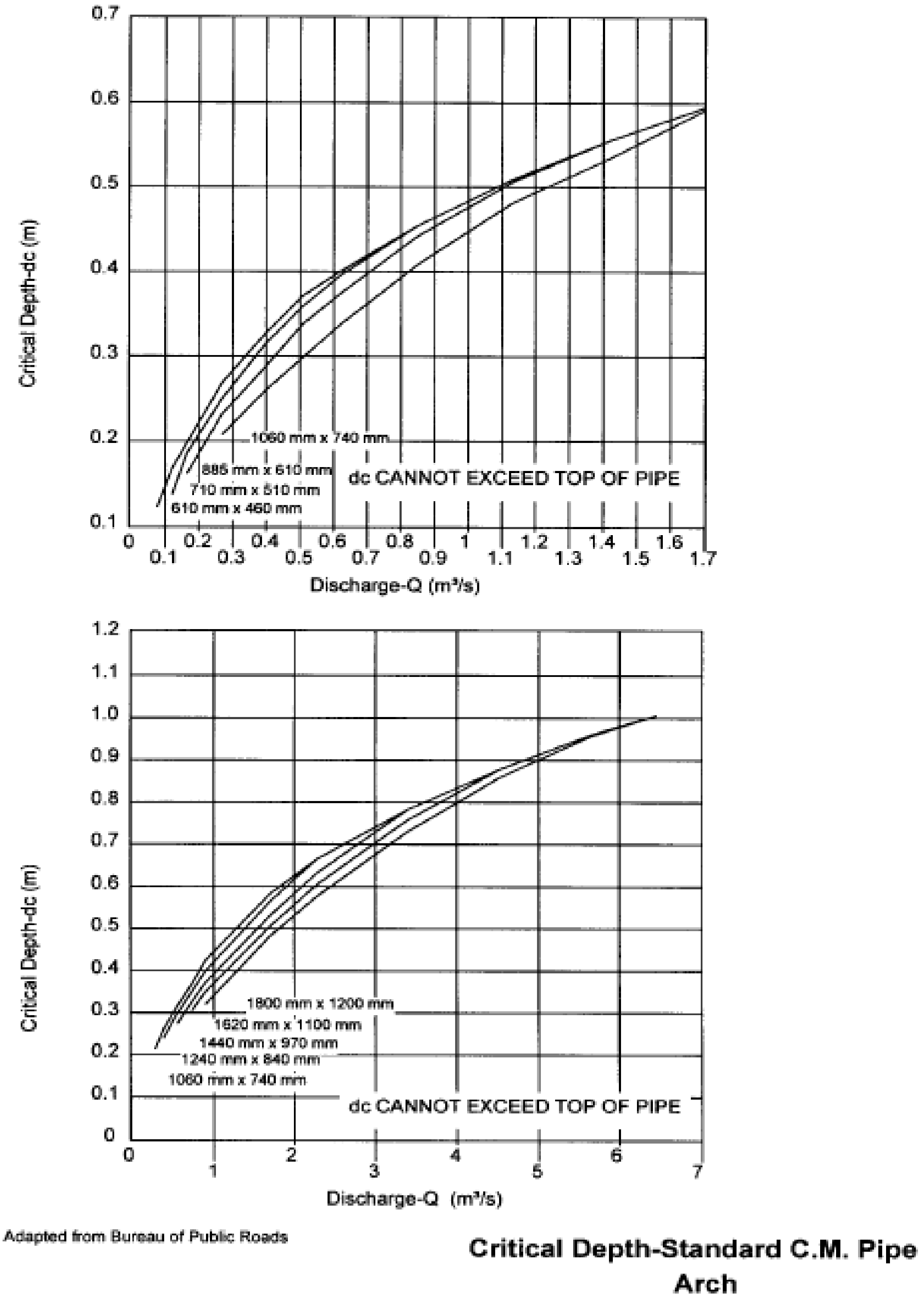


Figure 12.17: Outlet Control Nomograph: Critical Depth or Standard C.M. Pipe Arch

457mm Corner Radius

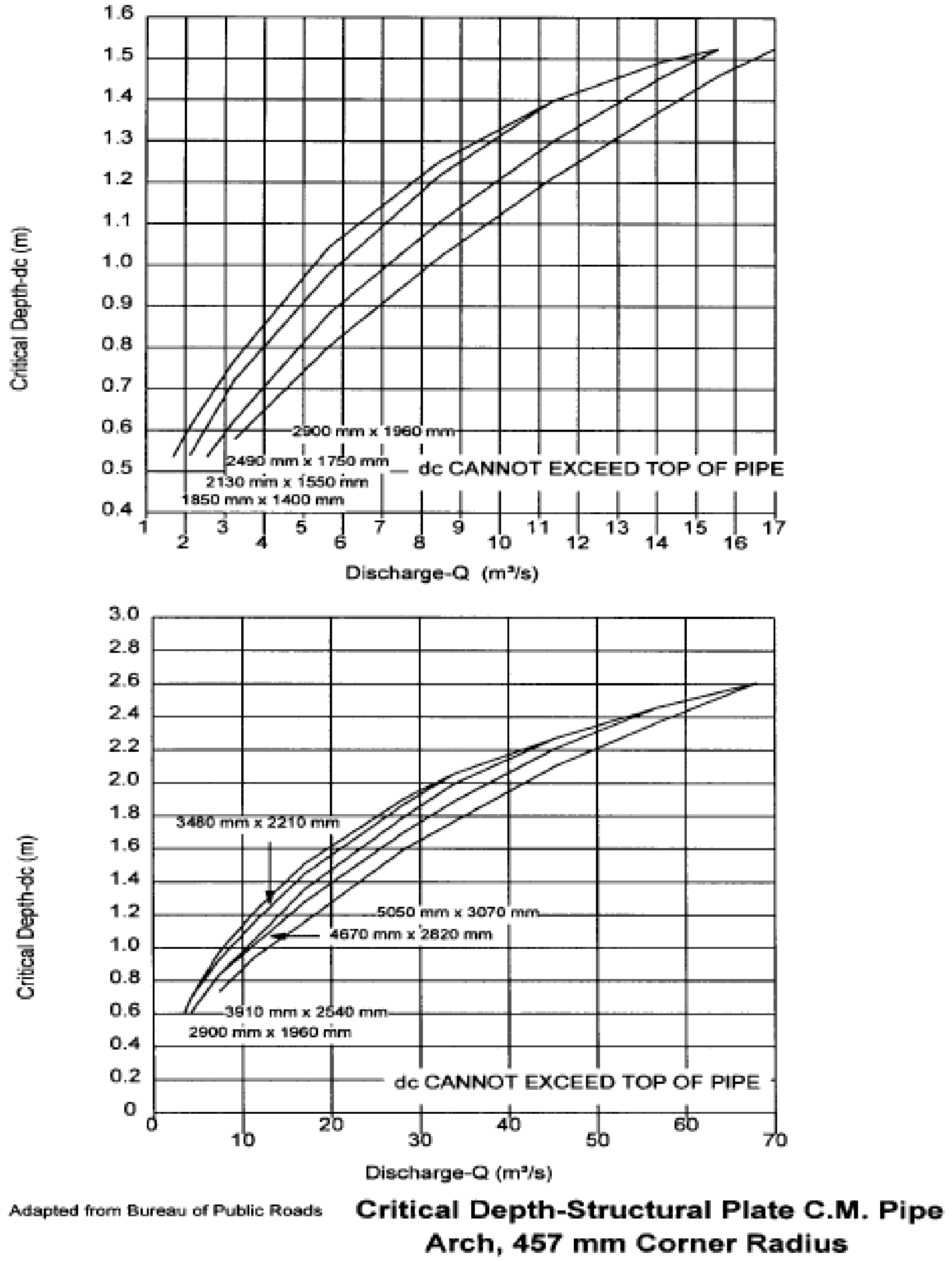
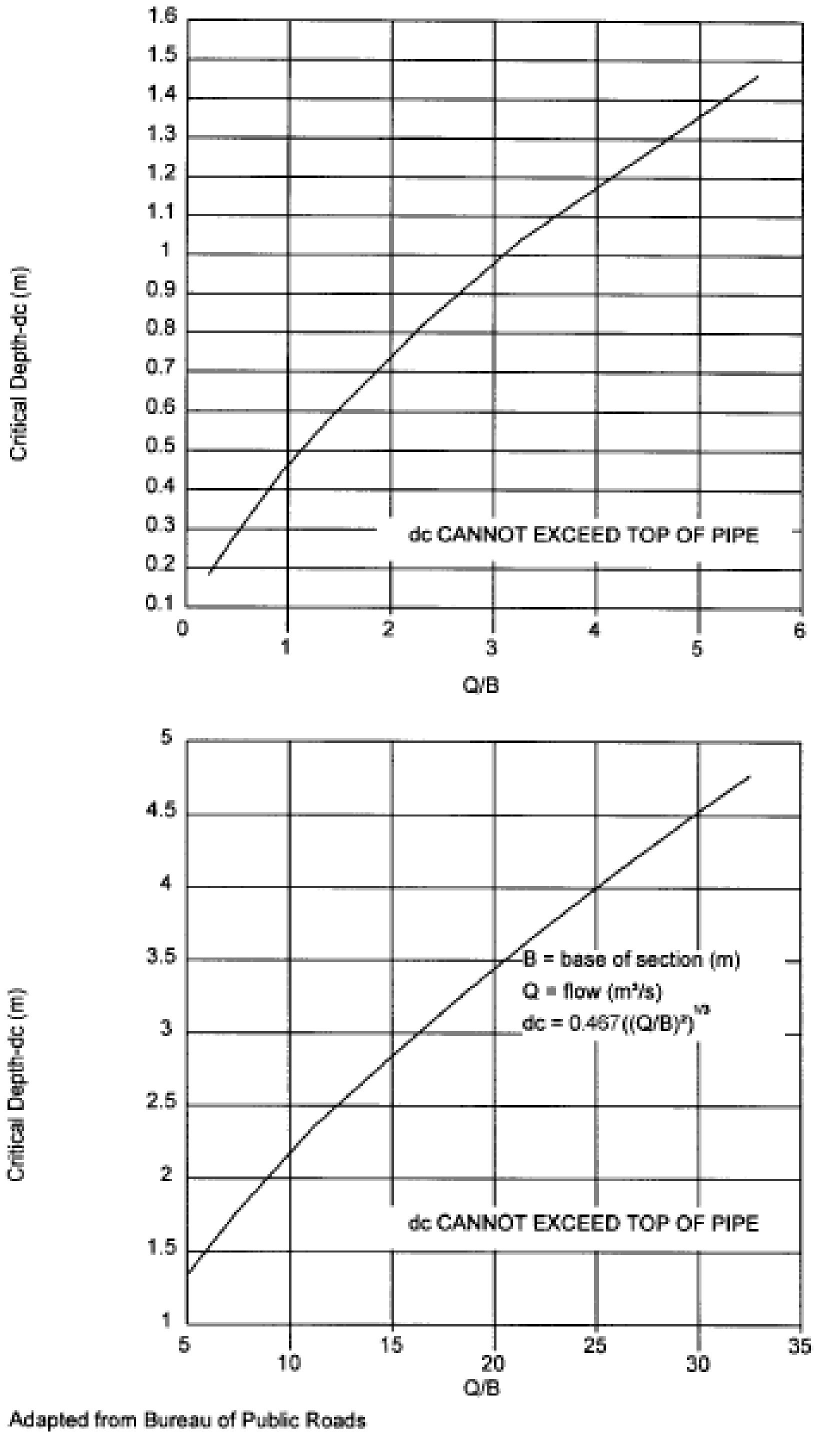


Figure 12.18: Outlet Control Nomograph: Critical Depth for C.M. Pipe Arch



Critical Depth-Rectangular Section

Figure 12.19: Outlet Control Nomograph: Critical Depth for Rectangular Sections

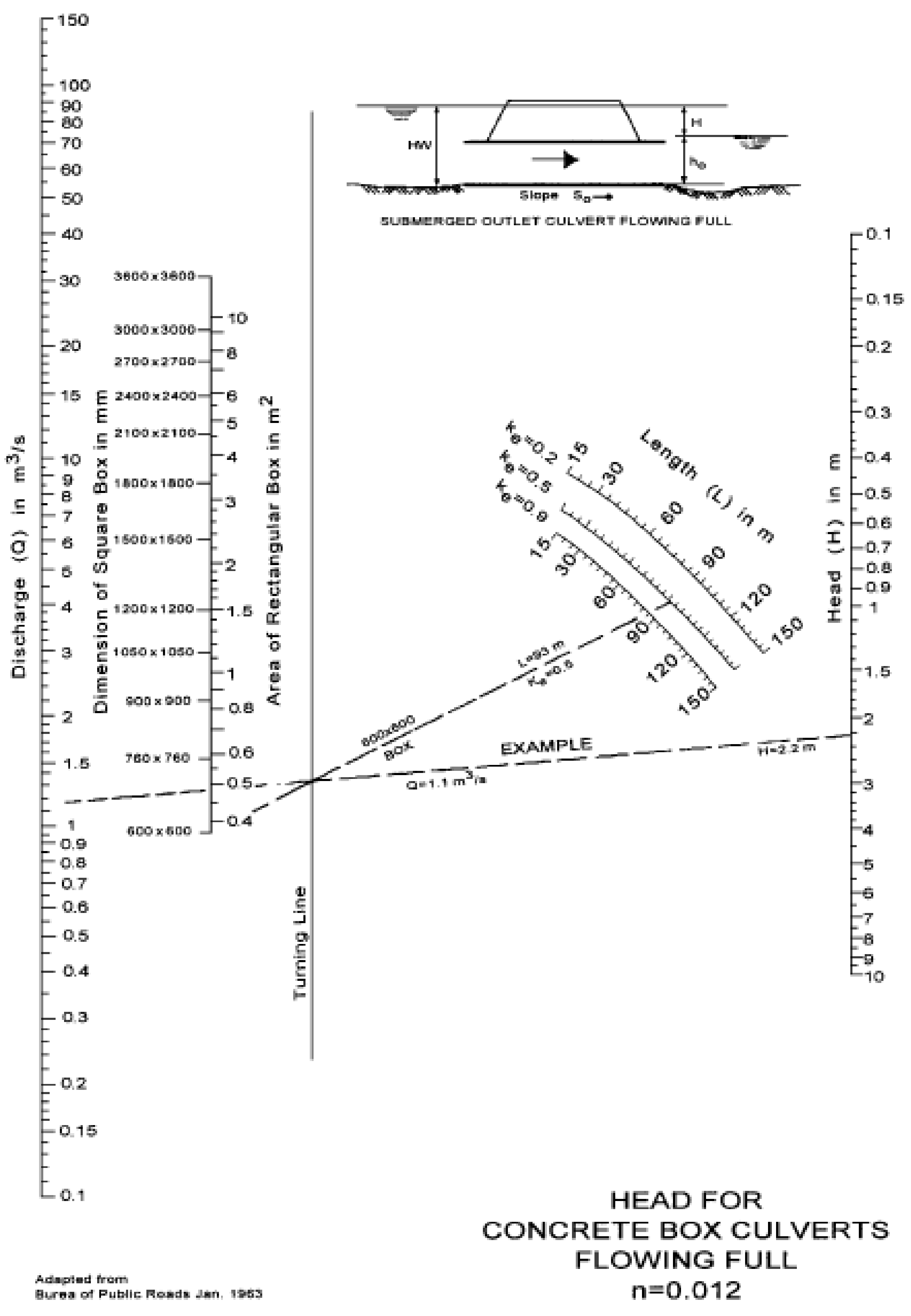


Figure 12.20: Outlet Control Nomograph – Box culverts flowing full

1:

13: Appendix 4: Fill Height Tables

Notes

a. Pipe Cover

The amount of cover over the top of a pipe is defined as the distance from the top of the crown of the pipe to the bottom of the pavement. It does not include any asphalt or concrete paving above the top course. The minimum amount of cover for most pipe materials is typically 0.6m except as described in the following paragraph. Unless the contract plans specify a specific pipe material, the designer should design for the schedule pipe fill heights as described in Division 7 of the *Standard Specifications*.

b. Shallow Cover Installation

Pipe systems should be designed to provide at least 0.6m of cover over the top of the pipe. This tends to provide adequate structural distribution of the live load and also allows a significant number of pipe alternatives to be specified on a contract. However, in some cases, it is not possible to lower a pipe profile to obtain the necessary minimum cover. In those cases, only concrete pipe of the class shown in table for Shallow Cover installations should be specified. Included in that table are typical pipe wall thicknesses for a given diameter. The pipe thickness must be taken into consideration in low cover applications. Justification must also be included in the hydraulic report describing why it was not possible to lower the pipe profile to obtain the preferred 0.6m of cover.

In addition to circular pipe, concrete box culverts and concrete arches are also available for use in shallow cover installations. The designer should consult with either the Regional Hydraulics Section/Contract or the HQ Hydraulics Engineer for additional guidance on the use of these structures in this application.

Table 13.1: Concrete Pipe (Metric)

Pipe Diameter mm	Maximum Cover in Metres				
	Plain AASHTO M86M	Class II AASHTO M170M	Class III AASHTO M170M	Class IV AASHTO M170M	Class V AASHTO M170M
300	5.5	3.0	4.3	6.5	7.9
450	5.5	3.4	4.3	6.5	8.5
600	5.0	3.4	4.6	6.5	8.5
750		3.4	4.6	7.0	9.0
900		3.4	4.6	7.0	9.0
1200		3.7	4.6	7.0	9.0
1500		3.7	4.9	7.5	9.0
1800		3.7	4.9	7.5	9.0
2100		3.7	4.9	7.5	9.0

Minimum Cover 0.6 metres

Table 13.2: Concrete Pipe for Shallow Cover Installations (Metric)

Pipe Diameter mm	Pipe Wall Thick mm	Maximum Cover in Metres			
		Plain AASHTO M86	Class III AASHTO M170	Class IV AASHTO M170	Class V AASHTO M170
300	50	0.45	0.45	0.30	0.15
450	63	0.45	0.45	0.30	0.15
600	75	0.45	0.45	0.30	0.15
750	88	0.45	0.45	0.30	0.15
900	100	0.45	0.45	0.30	0.15
1200	125		0.45	0.30	0.15
1500	150		0.45	0.30	0.15
1800	175		0.45	0.30	0.15
2100	200		0.45	0.30	0.15

Table 13.3: Fill Height Corrugated Steel Pipe (75 mm x 25 mm Corrugations)

Pipe Diameter mm	Shape	Maximum Cover in Metres				
		1.6 mm 16 ga	2.0 mm 14 ga	2.8 mm 12 ga	3.5 mm 10 ga	4.3 mm 8 ga
900	Circle	10.2	11.4	14.1	17.1	19.8
	Elong.	12.9	17.1	26.4	30.3	31.8
1050	Circle	8.1	9.0	10.8	12.6	14.4
	Elong.	10.8	14.7	21.6	25.2	27.3
1200	Circle	7.2	7.8	9.0	10.2	11.4
	Elong.	9.6	12.5	18.0	20.4	22.8
1350	Circle	6.6	6.9	7.8	8.4	9.3
	Elong.	8.4	11.4	15.6	16.8	18.6
1500	Circle	6.0	6.3	6.9	7.5	8.1
	Elong.	7.5	10.2	13.6	15.0	16.2
1650	Circle	5.7	6.0	6.6	6.9	7.5
	Elong.	6.9	9.3	13.2	13.8	15.0
1800	Circle	5.5	5.7	6.0	6.6	6.9
	Elong.	6.3	8.4	12.0	13.2	13.8
1950	Circle	5.2	5.5	6.0	6.3	6.3
	Elong.	5.2	7.8	12.0	12.6	12.6
2100	Circle	-	5.4	5.7	6.0	6.3
	Elong.	-	7.2	11.1	12.0	12.6

Pipe Diameter mm	Shape	Maximum Cover in Metres				
		1.6 mm 16 ga	2.0 mm 14 ga	2.8 mm 12 ga	3.5 mm 10 ga	4.3 mm 8 ga
2250	Circle	-	5.4	5.2	5.7	6.0
	Elong.	-	6.6	10.5	11.4	12.0
2400	Circle	-	-	5.4	5.7	5.7
	Elong.	-	-	9.9	11.4	11.4
2550	Circle	-	-	5.4	5.4	5.7
	Elong.	-	-	9.3	10.5	11.1
2700	Circle	-	-	5.4	5.4	5.4
	Elong.	-	-	8.7	9.9	10.5
2850	Circle	-	-	-	5.7	5.4
	Elong.	-	-	-	9.6	9.9
3000	Circle	-	-	-	5.7	5.4
	Elong.	-	-	-	8.7	9.6

Minimum cover 0.6m

Table 13.4: Fill Height Corrugated Steel Pipe (65mm x 12mm Corrugations)

Pipe Diameter mm	Shape	Maximum Cover in Metres				
		1.6 mm 16 ga	2.0 mm 14 ga	2.8 mm 12 ga	3.5 mm 10 ga	4.3 mm 8 ga
300	Circle	24.9	27.3	-	-	-
	Elong.	-	-	-	-	-
450	Circle	14.1	16.5	21.3	-	-
	Elong.	16.5	18.0	23.4	-	-
600	Circle	8.7	9.9	12.0	14.1	-
	Elong.	12.3	13.5	17.4	18.3	-
750	Circle	6.9	7.5	8.4	9.6	10.8
	Elong.	9.9	10.8	13.8	14.7	15.3
900	Circle	6.0	6.3	6.9	7.8	8.4
	Elong.	8.1	9.0	11.7	12.0	12.6
1050	Circle	5.7	6.0	6.3	5.7	7.2
	Elong.	9.0	17.0	17.6	11.4	14.4
1200	Circle	5.4	5.7	5.7	6.6	6.3
	Elong.	7.8	11.1	11.4	11.7	12.6
1350	Circle	-	5.4	5.7	6.0	6.0
	Elong.	-	9.9	11.4	12.0	12.0
1500	Circle	-	-	5.4	5.7	5.7
	Elong.	-	-	10.8	11.4	11.4

Pipe Diameter mm	Shape	Maximum Cover in Metres				
		1.6 mm 16 ga	2.0 mm 14 ga	2.8 mm 12 ga	3.5 mm 10 ga	4.3 mm 8 ga
1650	Circle	-	-	5.4	5.4	5.4
	Elong.	-	-	8.7	10.8	10.8
1800	Circle	-	-	-	5.4	5.4
	Elong.	-	-	-	8.7	10.8
1950	Circle	-	-	-	-	3.4
	Elong.	-	-	-	-	8.4
2100	Circle	-	-	-	-	5.1
	Elong.	-	-	-	-	6.9

Table 13.5: Fill Height Corrugated Aluminium Pipe (65mm x 12mm Corrugations)

Pipe Diameter mm	Shape	Maximum Cover in Metres				
		1.6 mm 16 ga	1.875mm 14 ga	2.625 mm 12 ga	3.375 mm 10 ga	4.100 mm 8 ga
300	Circle	13.5	13.5	23.4	-	-
	Elong.	-	-	-	-	-
450	Circle	8.1	9.0	10.8	-	16.8
	Elong.	9.0	-	15.6	12.6	-
600	Circle	6.3	6.6	7.5	8.1	11.1
	Elong.	6.6	-	11.7	12.0	12.6
750	Circle	-	5.4	6.3	6.6	7.2
	Elong.	-	-	9.3	9.6	9.9
900	Circle	-	4.5	5.7	6.0	6.3
	Elong.	-	-	7.8	8.1	8.4
1050	Circle	-	5.4	5.4	5.7	5.7
	Elong.	-	7.5	10.8	11.4	11.4
1200	Circle	-	-	5.4	5.4	6.4
	Elong.	-	-	8.45.1	10.8	10.8
1350	Circle	-	-	6.0	5.4	5.4
	Elong.	-	-	-	7.8	11.3
1500	Circle	-	-	-	5.1	5.1
	Elong.	-	-	-	5.4	6.9

Pipe Diameter mm	**Shape**	**Maximum Cover in Metres**				
		1.6 mm 16 ga	**1.875mm 14 ga**	**2.625 mm 12 ga**	**3.375 mm 10 ga**	**4.100 mm 8 ga**
1650	Circle	-	-	-	4.2	5.1
	Elong.	-	-	-	-	-
1800	Circle	-	-	-	-	3.9
	Elong.	-	-	-	-	

Table 13.6: Fill Height Corrugated Steel Structural Plate Pipe (150mm x 50mm Corrugations)

Pipe Diameter mm	Shape	Maximum Cover in Metres						
		12 ga	10 ga	8 ga	7 ga	5 ga	3 ga	1 ga
1500	Circle	12.6	15.0	17.4	18.9	21.3	23.7	26.4
	Elong.	-	18.6	24.3	27.9	33.6	39.6	43.2
1800	Circle	9.6	10.8	12.0	12.9	14.4	15.9	17.4
	Elong.	10.5	15.3	20.1	23.1	27.9	31.8	34.8
2100	Circle	7.8	8.7	9.6	9.9	10.8	11.7	12.9
	Elong.	9.0	13.2	17.1	19.8	21.6	23.4	25.8
2400	Circle	6.9	7.5	8.1	8.4	9.0	9.6	10.2
	Elong.	7.8	11.4	15.0	16.8	18.0	19.2	20.4
2700	Circle	6.3	6.6	7.2	7.5	7.8	8.1	8.7
	Elong.	6.9	10.2	13.5	15.0	15.6	16.2	17.4
3000	Circle	6.0	6.3	6.6	6.6	6.9	7.2	7.5
	Elong.	6.3	9.3	12.0	13.2	13.8	14.4	16.0
3300	Circle	6.3	6.0	6.3	6.3	6.6	6.9	7.2
	Elong.	-	8.4	10.8	12.6	13.2	13.8	14.4
3600	Circle	5.1	5.7	6.0	6.0	6.3	6.3	6.6
	Elong.	-	7.5	9.9	11.4	12.6	12.6	13.2
3900	Circle	4.8	5.4	5.7	5.7	6.0	6.0	6.3
	Elong.	-	6.9	9.2	10.5	12.0	12.0	12.6

Pipe Diameter mm	Shape	Maximum Cover in Metres						
		12 ga	10 ga	8 ga	7 ga	5 ga	3 ga	1 ga
4200	Circle	4.5	5.4	5.4	5.7	5.7	5.7	6.0
	Elong.	-	6.6	8.4	9.9	11.4	11.4	12.0
4500	Circle	4.2	5.4	5.4	5.4	5.7	5.7	5.7
	Elong.	-	6.0	8.1	9.3	11.1	11.4	11.4
4800	Circle	-	5.4	5.4	5.4	5.4	5.4	5.7
	Elong.	-	5.7	7.5	8.7	10.4	10.8	10.4
5100	Circle	-	5.1	5.4	5.4	5.4	5.4	5.4
	Elong.	-	5.4	6.0	8.1	9.6	10.8	10.8
5400	Circle	-	-	5.1	5.4	5.4	5.4	5.4
	Elong.	-	-	6.6	7.5	9.3	10.8	10.8
5700	Circle	-	-	5.1	5.1	5.4	5.4	5.4
	Elong.	-	-	6.3	6.9	8.1	9.3	10.5
6000	Circle	-	-	-	5.1	5.1	5.4	5.4
	Elong.	-	-	-	6.0	6.9	7.8	9.0
6300	Circle	-	-	-	-	5.1	5.1	5.4
	Elong.	-	-	-	-	6.0	6.9	7.8

Table 13.7: Fill Height Corrugated Aluminium Structural Plate Pipe (225mm x 60mm Corrugations)

Pipe Diameter mm	Shape	Maximum Cover in Metres							
		2.25	2.5	3.125	3.75	4.375	5.0	5.625	6.25
1800	Circle	5.4	6.0	8.1	8.4	9.0	9.6	10.5	10.8
	Elong.	-	-	2.7	11.1	13.2	15.0	16.5	18.0
1950	Circle	5.4	6.0	7.2	7.8	8.4	8.7	9.3	9.6
	Elong.	-		7.8	10.2	12.0	13.8	15.0	16.5
2100	Circle	4.5	5.4	6.9	7.2	7.5	8.1	8.4	8.7
	Elong.	-		7.2	9.3	11.1	12.6	16.1	15.3
2250	Circle	4.2	5.1	6.6	6.9	7.2	7.5	7.8	8.1
	Elong.	-		6.9	8.7	10.5	12.0	13.2	14.4
2400	Circle	3.9	4.8	6.3	6.6	6.6	6.9	7.2	7.5
	Elong.	-		-	8.1	9.9	11.1	12.3	13.5
2550	Circle	3.9	4.5	6.0	6.3	6.3	6.6	6.9	5.2
	Elong.	-			7.3	9.3	10.5	11.4	12.6
2700	Circle	3.6	4.2	5.7	6.0	6.3	6.3	6.6	6.6
	Elong.	-			7.2	8.7	9.9	10.8	12.0
2850	Circle	3.3	3.9	5.4	6.0	6.0	6.3	6.3	6.6
	Elong.	-			6.9	8.1	9.3	10.2	11.4
3000	Circle	-	3.9	5.1	5.7	6.0	6.0	6.0	6.3
	Elong.	-			6.6	7.8	9.0	9.9	10.4

Pipe Diameter mm	Shape	Maximum Cover in Metres							
		2.25	2.5	3.125	3.75	4.375	5.0	5.625	6.25
3150	Circle	-	3.6	4.8	5.7	5.7	6.8	6.0	6.0
	Elong.	-			6.3	7.5	8.4	9.3	10.2
3300	Circle	-	3.6	4.5	5.7	5.7	5.7	5.7	6.0
	Elong.	-			6.0	7.7	8.1	9.0	9.6
3450	Circle	-		4.5	5.4	5.7	5.7	5.7	5.7
	Elong.	-			5.7	6.6	7.0	8.4	9.3
3600	Circle	-		4.2	5.4	5.4	5.7	5.7	5.7
	Elong.	-			-	6.6	7.5	8.1	9.0
3750	Circle	-		3.9	5.1	5.4	5.4	5.7	5.7
	Elong.	-				6.3	7.2	7.8	8.4
3900	Circle	-		3.9	5.1	5.4	5.4	5.4	5.7
	Elong.	-				6.0	6.9	7.5	8.1
4050	Circle	-			4.8	5.4	5.4	5.4	5.4
	Elong.	-				5.7	6.6	7.2	7.8
4200	Circle	-			4.5	5.4	5.4	5.4	5.4
	Elong.	-					6.3	6.9	7.5
4350	Circle	-				5.4	5.4	5.4	5.4
	Elong.	-					6.0	6.6	7.2
4500	Circle	-				5.1	5.4	5.4	5.4
	Elong.	-					6.0	6.6	7.2

Minimum cover 0.6 m

Table 13.8: Fill Height Corrugated Steel Straight Plate Pipe Arch (150mm x 50mm Corrugations)

Span x Rise (m) x (m)	Min Allow gauge	Cor Radius (mm)	Max Allowable Cover (m) for corner Pressures of		Min Cover (m)
			210kN/m^2	320kN/m^2	
1.9 x 1.4	12	450	5	7	
2.2 x 1.6	12	450	4	6	
2.4 x 1.7	12	450	4	5	
2.7 x 1.9	12	450	3	5	
3.0 x 2.0	12	450	3	5	
3.4 x 2.2	12	450	3	4	
3.7 x 2.4	12	450	2	4	
4.0 x 2.6	12	450	2	3	
4.4 x 2.7	12	450	2	3	
4.7 x 2.9	10	450	2	3	
4.9 x 3.0	10	450	2	3	
5.1 x 3.1	10	450	2	3	
4.1 x 2.9	12	775	2	6	
4.4 x 3.0	12	775	4	5	
4.7 x 3.2	10	775	3	5	
5.0 x 3.3	10	775	3	5	
5.3 x 3.5	10	775	3	5	
5.6 x 3.7	8	775	3	4	

Span x Rise (m) x (m)	**Min Allow gauge**	**Cor Radius (mm)**	**Max Allowable Cover (m) for corner Pressures of**		**Min Cover (m)**
			210kN/m^2	**320kN/m^2**	
6.0 x 3.8	8	775	2	4	
6.2 x 3.9	8	775	2	4	
6.4 x 4.1	7	775	2	4	

Table 13.9: Fill Height Corrugated Steel Straight Plate Pipe Arch (225mm x 62mm Corrugations)

Span x Rise (m) (m)	Min Allow Thk (mm)	Max Allowable Cover (m) for corner Pressures of		Min Cover (m)
		210kN/m^2	320kN/m^2	
1.8 x 1.7	2.5	7	7	
2.1 x 1.8	2.5	6	6	
2.5 x 1.9	2.5	5	5	
2.8 x 2.1	2.5	4	4	
3.1 x 2.2	2.5	4	4	
3.5 x 2.3	2.5	3	3	
3.8 x 2.5	2.5	3	3	
4.0 x 2.6	3.1	3	4	
4.4 x 2.7	3.8	3	5	
4.7 x 2.9	3.8	3	4	
5.0 x 3.0	4.4	3	4	
5.3 x 3.1	5.0	3	4	

Table 13.10: 68 mm x 13 mm Corrugations – Corrugated Steel Pipe (AASHTO M196 M)

Pipe Diameter mm	**Maximum Cover in Metres**				
	1.6 mm 16 ga	**2.0 mm 14 ga**	**2.8 mm 12 ga**	**3.5 mm 10 ga**	**4.3 mm 8 ga**
300	30.5	30.5	30.5	30.5	
450	30.5	30.5	30.5	30.5	
600	30.0	30.5	30.5	30.5	30.5
750	24.0	30.0	30.5	30.5	30.5
900	20.0	24.5	30.5	30.5	30.5
1050	17.0	21.5	30.0	30.5	30.5
1200	15.0	18.5	26.0	30.5	30.5
1350		16.5	23.0	30.0	30.5
1500			21.0	27.0	30.5
1650				24.5	30.0
1800				22.5	27.5
1950					24.5
2100					21.0

Table 13.11: 75 mm x 25 mm Corrugations – Aluminium Steel Pipe (AASHTO M196 M)

Pipe Diameter mm	Maximum Cover in Metres				
	1.5 mm 16 ga	1.9 mm 14 ga	2.7 mm 12 ga	3.4 mm 10 ga	4.2 mm 8 ga
900	13.0	20.0	23.0	30v	
1050	11.0	14.0	20.0	25.5	
1200	9.5	12.0	17.5	22.0	27.5
1350	8.5	10.5	15.0	20.0	24.5
1500		9.5	13.5	17.5	22.0
1650		8.5	12.5	16.0	20.0
1800		8.0	11.5	14.5	18.0
1950		7.5	10.5	13.5	17.0
2100			9.5	12.5	15.5
2250			9.0	11.5	14.5
2400			8.0	11.0	13.5
2550				10.0	12.5
2700				9.5	12.0
2850					11.5
3000					10.5

Table 13.12: Aluminium Pipe Arch 68 mm X 13 mm Corrugations (AASHTO M196 M)

Span – Rise mm mm	Corner Radius mm	Thickness		Min Cover Metres	Maximum Cover in Feet for Soil Bearing Capacity of:	
		Mm	Gage		190 kPa	290 kPa
430 X 330	75	1.5	16 ga	0.6	3.7	5.5
530 X 380	75	1.5	16 ga	0.6	3.0	4.3
610 X 460	75	1.5	16 ga	0.6	2.1	4.0
710 X 510	75	1.9	14 ga	0.6	1.5	3.5
885 X 610	75	1.9	14 ga	0.8	NS	2.1
1060 X 740	89	2.7	12 ga	0.8	NS	2.1
1240 X 840	102	2.7	12 ga	0.8	NS	1.8
1440 X 970	127	3.4	10 ga	0.8	NS	2.0
1620 X 1100	152	3.4	10 ga	0.8	NS	2.7
1800 X 1200	178	4.2	8 ga	0.6	NS	3.7

NS = Not Suitable

Table 13.13: Corrugated Steel Pipe Arch 68 mm X 13 mm Corrugations (AASHTO M36 M)

Span x Rise mm mm	Corner Radius mm	Thickness		Min Cover Metres	Maximum Cover in Feet for Soil Bearing Capacity of:	
		Mm	Gage		190 kPa	290 kPa
430 X 330	75	1.6	16 ga	0.6	3.7	5.5
530 X 380	75	1.6	16 ga	0.6	3.0	4.3
610 X 460	75	1.6	16 ga	0.6	2.1	4.0
710 X 510	75	1.6	16 ga	0.6	1.5	3.4
885 X 610	75	1.6	16 ga	0.8	NS	2.1
1060 X 740	88	1.6	16 ga	0.8	NS	2.1
1240 X 840	100	2	14 ga	0.8	NS	1.8
1440 X 970	125	2.8	12 ga	0.8	NS	2.4
1620 X 1100	150	2.8	12 ga	0.8	NS	2.7
1800 X 1200	175	3.5	10 ga	0.6	NS	3.0
1950 X 1320	200	4.3	8 ga	0.6	1.5	2.0
2100 X 1450	225	4.3	8 ga	0.6	1.5	3.0

NS = Not Suitable

Table 13.14: Corrugated Steel Pipe Arch 75 mm X 25 mm Corrugations (AASHTO M36 M)

Span x Rise mm mm	Corner Radius mm	Thickness		Min Cover Metres	Maximum Cover in Feet for Soil Bearing Capacity of:	
		Mm	Gage		190 kPa	290 kPa
1010 X 790	125	2	14 ga	0.8	2.4	3.7
1160 X 920	150	2	14 ga	0.6	2.4	4.0
1340 X 1050	175	2	14 ga	0.6	2.4	4.0
1520 X 1170	200	2	14 ga	0.6	2.4	4.0
1670 X 1300	225	2	14 ga	0.8	2.7	4.0
1850 X 1400	300	2	14 ga	0.8	3.4	4.9
2050 X 1500	350	2	14 ga	0.8	3.4	5.2
2200 X 1620	350	2	14 ga	0.8	3.0	4.9
2400 X 1720	400	2	14 ga	0.8	3.4	5.2
2600 X 1820	400	2.8	12 ga	0.6	3.0	4.5
2840 X 1920	450	2.8	12 ga	0.6	3.0	4.9
2970 X 2020	450	2.8	12 ga	0.6	3.0	4.5
3240 X 2120	450	3.5	10 ga	0.6	2.7	4.3
3470 X 2220	450	3.5	10 ga	0.6	2.4	4.4
3600 X 2320	450	4.3	8 ga	0.6	2.1	3.7

Table 13.15: Corrugated Steel Structural Plate Pipe Arch (152 mm x 51 mm Corrugations)

Span – Rise mm mm	Corner Radius mm	Thickness		190 kPa Soil Bearing Capacity		290 kPa Soil Bearing Capacity	
		Mm	Gage	Min Cover m	Max Cover m	Min Cover m	Max Cover m
1850 X 1400	457	2.8	12 ga	0.6	5.0	0.6	7.0
2130 X 1550	457	2.8	12 ga	0.6	4.3	0.6	6.5
2410 X 1700	457	2.8	12 ga	0.6	4.0	0.6	6.0
2690 X 1850	457	2.8	12 ga	0.6	3.4	0.6	5.0
2970 X 2010	457	2.8	12 ga	0.6	3.0	0.6	4.5
3330 X 2160	457	2.8	12 ga	0.6	2.7	0.6	4.3
3610 X 2310	457	2.8	12 ga	0.6	2.1	0.6	4.0
3910 X 2540	457	2.8	12 ga	0.8	1.8	0.6	7.7
4040 X 2840	787	2.8	12 ga	0.6	4.0	0.6	5.0
4320 X 3000	787	2.8	12 ga	0.6	3.7	0.6	5.0
4670 X 3150	787	3.5	10 ga	0.6	3.4	0.6	4.5
4950 X 3300	787	3.5	10 ga	0.6	3.4	0.6	4.3
5230 X 3450	787	3.5	10 ga	0.8	3.0	0.8	4.0
5510 X 3610	787	4.5	8 ga	0.8	3.0	0.8	3.7
5870 X 3760	787	4.5	8 ga	0.8	2.7	0.8	4.0
6070 X 3910	787	4.8	6 ga	0.8	2.7	0.8	4.0
6270 X 4010	787	4.8	6 ga	0.9	2.1	0.9	4.0

Table 13.16: Aluminium Structural Plate Arch

(230 mm X 64 mm Corrugations, 19 mm steel bolts, 4 bolts/corrugation)

Span – Rise mm mm		Corner Radius mm	Minimum Gage Thickness mm	Minimum Cover m	Maximum Cover (1) in metres for Soil Bearing Capacity of:	
					190 kPa	290 kPa
a	1800 X 1650	808	2.5	0.6	6.0*	7.0*
b	2100 X 1750	808	2.5	0.6	5.0*	6.5*
c	2210 X 1800	808	2.5	0.6	6.0*	6.0*
d	2360 X 1830	808	2.5	0.6	5.5*	5.5*
e	2570 X 1910	808	2.5	0.6	5.0*	5.0*
f	2820 X 1960	808	2.5	0.6	4.5*	4.5*
g	3120 X 2060	808	2.5	0.6	4.3*	4.3*
h	3280 X 2080	808	2.5	0.6	4.0*	4.0*
i	3480 X 2160	808	2.5	0.6	3.7*	3.7*
j	3840 X 2260	808	3.2	0.6	4.3	5.0*
k	3940 X 2290	808	3.8	0.6	4.0	4.3*
l	3990 X 2490	808	3.8	0.6	4.0	5.5*
m	4240 X 2570	808	3.8	0.6	3.7	5.0*
n	4470 X 2950	808	4.4	0.6	3.7	5.5
o	4670 X 3050	808	4.4	0.6	3.4	5.0
p	4900 X 3150	808	5.1	0.6	3.0	5.0
q	5110 X 3250	808	5.1	0.67	3.0	4.5

Span – Rise mm mm		Corner Radius mm	Minimum Gage Thickness mm	Minimum Cover m	Maximum Cover (1) in metres for Soil Bearing Capacity of:	
					190 kPa	290 kPa
r	5260 X 3350	808	5.7	0.69	3.0	4.5
s	5490 X 3450	808	6.4	0.69	2.7	4.3
t	5690 X 3560	808	6.4	0.71	2.7	4.3

* Fill limited by the seam strength of the bolts

(1) Additional sizes and varying cover heights are available, depending on gage thickness and reinforcement spacing. Contact the OSC Hydraulics Branch for more information.

Table 13.17: Corrugated Steel Structural Plate Circular Pipe

152 mm x 51 mm Corrugations

Pipe Diameter mm	Minimum Cover m	Maximum Cover in Metres						
		2.8 mm 12 ga	3.5 mm 10 ga	4.5 mm 8 ga	4.8 mm 7 ga	5.5 mm 5 ga	6.5 mm 3 ga	7.0 mm 1 ga
1500	0.6	13	19	25.5	28	30.5	30.5	30.5
1800	0.6	10.5	16	21	24	28.5	30.5	30.5
2100	0.6	9	13.5	18	20.5	24.5	29	30.5
2400	0.6	8	12	16	18	21.5	22.5	28
2700	0.6	7	10.5	14	16	19.5	23	24.5
3000	0.6	6.5	9.5	13	14.5	17.8	20.5	22.5
3300	0.6	6	9	11.5	13	16	18.5	20
3600	0.6	5.5	8	11.5	12	14.5	17	18.5
3900	0.6	5	7	9.5	11	13	16	17
4200	0.6	4.5	6.5	9	10	12.5	14.5	16
4500	0.6	4.3	6	8.5	9.5	11.5	13.5	15
4800	0.6		6	8	9	10.5	13	14
5100	0.9		5.5	7	8.5	10	12	13
5400	0.9			7	8	9.5	11.5	12.5
5700	0.9				7.5	9	10.5	12
6000	0.9				7	9	10	11.5

Table 13.18: Circular Aluminium Structural Plate 230 mm x 64 mm in Corrugations with Galvanized Steel Bolts

Pipe Diameter mm	Maximum Cover in Metres						
	2.5 mm	3.2 mm	3.8 mm	4.4 mm	5.1 mm	5.7 mm	6.4 mm
1500	9.5	13.5	18.5	21.5	24.5	28	28
1800	7.5	11.5	15	17.5	20.5	23.5	23.5
2100	6.5	10	13	15	17.5	20	20
2400	6	8.5	11.5	13.5	15	17.5	17.5
2700	5	7.5	10	12	13.5	15.5	15.5
3000	4.5	6.5	9	10.5	12	14	14
3300	4.3	6	8	10	11.5	13	13
3600	3.7	5.5	7.5	9	10	11.5	11.5
3900		5	7	8	9.5	10.5	10.5
4200			6.5	7.5	9	10	10
4500				7	8	9	9

Table 13.19: Aluminium Pipe Arch 75 mm X 25 mm Corrugations (AASHTO M196 M)

Span x Rise mm mm	Corner Radius mm	Thickness		Minimum Cover Metres	Maximum Cover in Feet for Soil Bearing Capacity of:	
		Mm	Gage		190 kPa	290 kPa
1010 x 790	127	1.9	14 ga	0.8	2.4	3.7
1160 x 920	152	1.9	14 ga	0.6	2.4	4
1340 x 1050	178	1.9	14 ga	0.6	2.4	4
1520 x 1170	203	1.9	14 ga	0.6	2.4	4
1670 x 1300	229	1.9	14 ga	0.6	2.7	4
1850 x 1400	305	1.9	14 ga	0.6	3.4	5
2050 x 1500	356	1.7	12 ga	0.6	3.4	5
2200 x 1620	356	2.7	12 ga	0.6	3	5
2400 x 1720	406	2.7	12 ga	0.6	3.4	5
2600 x 1820	406	3.4	10 ga	0.6	3	4.5
2840 x 1920	457	4.2	8 ga	0.6	3	5

14: Appendix 5: Supporting References & Publications

This appendix provides an outline of selected reference publications and documents from highway and road authorities in other countries. They have been listed here as supplementary design references to assist engineers with specialist design problems, as well as for general background information. All of the listed publications are available for download free of charge from the Internet. Designers in Nigeria using these publications should bear in mind that the standards presented in these publications may differ from those presented in Volume IV of the Nigeria Highway Design Manual and therefore should be used with appropriate care and engineering judgement.

Table 14.1: Recommended supplementary references for drainage design

Ref No:	Title	Author	Description	Pub No.	Year:	PDF Download Link:
1	Urban Drainage Design Manual Third Edition	FHWA	The Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22 (HEC-22), provides a comprehensive and practical guide for the design of storm drainage systems associated with transportation facilities. Design guidance is provided for the design of storm drainage systems which collect, convey, and discharge stormwater flowing within and along the highway right-of-way. Methods and procedures are given for the hydraulic design of storm drainage systems. Design methods are presented for evaluating rainfall and runoff magnitude, pavement drainage, gutter flow, inlet design, median and roadside ditch flow, structure design, and storm drain piping. Procedures for the design of detention facilities are also presented, along with an overview of stormwater pumping stations and urban water quality practices. This edition includes major revisions to chapters 4 and 7.	NHI-10-009	2009	http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf
2	Design of Roadside Channels with Flexible Linings, Third Edition	FHWA	Flexible linings provide a means of stabilizing roadside channels. Flexible linings are able to conform to changes in channel shape while maintaining overall lining integrity. Long-term flexible linings such as riprap, gravel, or vegetation (reinforced with synthetic mats or unreinforced) are suitable for a range of hydraulic conditions. Unreinforced vegetation and many transitional and temporary linings are suited to hydraulic conditions with moderate shear stresses. Design procedures are given for four major categories of flexible lining: vegetative linings; manufactured linings (RECPs); riprap, cobble, gravel linings; and gabion mattress linings. Design procedures for composite linings, bends, and steep slopes are also provided. The design procedures are based on the concept of maximum permissible tractive force. Methods for determination of hydraulic resistance applied shear stress as well as permissible shear stress for individual linings and lining types are presented.	IF-05-114	2005	http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf
3	Hydraulic Design of Energy Dissipators for Culverts and Channels	FHWA	The purpose of this circular is to provide design information for analyzing and mitigating energy dissipation problems at culvert outlets and in open channels. The first three chapters provide general information on the overall design process (Chapter 1), erosion hazards (Chapter 2), and culvert outlet velocity and velocity modification (Chapter 3). These provide a background and framework for anticipating dissipation problems. In addition to describing the overall design process, Chapter 1 provides design examples to compare selected energy dissipators. The next three chapters provide assessment tools for considering flow transitions (Chapter 4), scour (Chapter 5), and hydraulic jumps (Chapter 6).	NHI-06-086	2006	http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/

			For situations where the tools in the first six chapters are insufficient to fully mitigate a dissipation problem, the remaining chapters address the design of six types of constructed energy dissipators. Although any classification system for dissipators is limited, this circular uses the following breakdown: internal (integrated) dissipators (Chapter 7), stilling basins (Chapter 8), streambed level dissipators (Chapter 9), riprap basins and aprons (Chapter 10), drop structures (Chapter 11), and stilling wells (Chapter 12).			
4	Hydraulic Design of Highway Culverts, Third Edition	FHWA	Hydraulic Design Series Number 5 (HDS 5) originally merged culvert design information contained in Hydraulic Engineering Circulars (HEC) 5, 10, and 13 with other related hydrologic, storage routing and special culvert design information. This third edition is the first major rewrite of HDS 5 since 1985, updating all previous information and adding new information on software solutions, aquatic organism passage, culvert assessment, and culvert repair and rehabilitation. The result is a comprehensive culvert design publication. The appendices of the publication contain the equations and methodology used in developing the design charts (nomographs) and software programs, information on hydraulic resistance of culverts, the commonly used design charts, and Design Guidelines (DG) illustrating various culvert design calculation procedures. The number of design charts provided has been reduced recognizing the increased use of software solutions; however, the full set of culvert design charts will continue to be available in the archived second edition of HDS 5.	HIF-12-026	2012	http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf
5	Highway Hydrology Second Edition	FHWA	Highway Hydrology, Hydraulic Design Series No. 2 (HDS-2), discusses the physical processes of the hydrologic cycle that are important to highway engineers. These processes include the approaches, methods and assumptions applied in design and analysis of highway drainage structures. Hydrologic methods of primary interest are frequency analysis for analyzing rainfall and ungaged data; empirical methods for peak discharge estimation; and hydrograph analysis and synthesis. The document describes the concept and several approaches for determining time of concentration. The peak discharge methods discussed include log Pearson type III, regression equations, the SCS graphical method (curve number method), and rational method. The technical discussion of each peak flow approach also includes urban development applications. The document presents common storage and channel routing techniques related to highway drainage hydrologic analyses. The document describes methods used in the planning and design of stormwater management facilities. Special topics in hydrology include discussions of arid lands hydrology, wetlands hydrology, snowmelt hydrology, and hydrologic modeling, including geographic information system approaches and applications.	NHI-02-001	2002	http://isddc.dot.gov/OLPFiles/FHWA/013248.pdf

			This edition also includes new sections on wetlands hydrology and snowmelt hydrology, an expanded section on arid lands hydrology, and corrections of minor errors, and inclusion of dual units.			
6	Highways in the Coastal	FHWA	The HEC-25 provides guidance for the planning, analysis, design, and operation of highways and bridges in the coastal environment. HEC-25	NHI-07-096	2008	http://www.fhwa.dot.gov/engineering/hydraulics/pubs/07096/07096.p
7	Culvert Design for Aquatic Organism Passage	FHWA	This document presents a stream simulation design procedure, methods and best practices for designing culverts to facilitate aquatic organism passage (AOP). The primary goal of this document is to incorporate many of the current geomorphic-based design approaches for AOP while providing a procedure based on quantitative best practices. It presents a bed stability-based approach that accounts for the physical processes related to the natural hydraulic, stream stability, and sediment transport characteristics of a particular stream crossing. Specific information on fish, or other aquatic organisms, is not required, but should be incorporated when required. The document provides a context for stream crossing design and describes the applicability of the design procedure. It also provides important background information a designer should be familiar with including how culverts create barriers, techniques for culvert assessments and inventories, fish biology, fish passage hydrology, stream geomorphology, construction, and post-construction. Detailed technical information supporting the practices used within the design procedure and several design examples are included in the appendices.	HIF-11-008	2010	http://www.fhwa.dot.gov/engineering/hydraulics/pubs/11008/hif11008.pdf
8	Design Manual	UK Highways	Section 2 – Drainage	Various	Various	http://www.dft.gov.uk/ha/standard
			This section of the UK Highways Agency Design Manual for Roads and Bridges includes a range of advice notes covering design of drainage elements associated with highways in the UK. Standards and procedures have been developed for UK conditions and therefore care should be exercised when using this reference for Nigeria-based projects.			
			Part 1:			
			Design of Outfalls for Surface Water Channels			
			Edge of Pavement Details			
			Vegetative Treatment Systems for Highway Runoff			
			Drainage of Runoff from Natural Catchments			
			Part 2:			
			Surface Drainage of Wide Carriageways			
			Part 3:			
			Surface and Sub-surface Drainage Systems for Highways			
			Spacing of Road Gullies			