COMITRAN
CONSEJO SECTORIAL DE MINISTROS DE TRANSPORTE DE CENTROAMÉRICA

SIECA
SECRETARÍA DE INTEGRACIÓN ECONÓMICA CENTROAMERICANA

Hidrologic and Hydraulic Technical Considerations Manual

for Road Infrastructure in Central America

First Edition 2016

This edition was made by the support of
Climate Change Adaptation and Strategic
Risk Management Department (DACGER) of
Ministry of Public Works, Transportation,
Housing and Urban Development (MOPTVDU).

First Edition

El Salvador, C.A., 2016

PRESENTATION

In 1997, the Sectoral Council of Central American Transport Ministers, COMITRAN, approved the document "Transport Sector for the Central American competiveness and integration", which, amongst its recommendations, it included the need to generate and harmonize innovative technical regulations that allow this region to improve and ensure people's and goods' transit in the Central American Isthmus.

In that context, the Council has prioritized within its strategic working areas, the issue of public infrastructure adaptation to Climate Change, in order to increase resilience of those works before the constant threat of extreme natural phenomena that periodically occur in the region. Thus, such Council states, develops and promotes a variety of structural (infrastructure) and non-structural measures, these last ones framed in a set of regulations that reduce the roadworks vulnerability, that ensure an optimal connectivity and development of the countries of the region.

Based on the above, in the thirty-third ordinary meeting of COMITRAN held in August 2014 in Managua, Nicaragua, the Ministers agreed on developing, in the Central American framework, a new regulation that incorporates hydrologic and hydraulic guidelines for the regional road infrastructure planning, design, construction and maintenance, in order to reduce the road vulnerability against hydro meteorological phenomena; and instruct the Central American Economic Integration Secretariat, to start these works.

This is how SIECA, and the Climate Change Adaptation and Strategic Risk Management Office (DACGER) from the Ministry of Public Works of El Salvador, accompanied the management of development of this regulation, and along with the valuable support of the Japan International Cooperation Agency (JICA) the technical and financial support required for the development of this Manual, was possible.

It is important to mention that JICA has been supporting the region on issues related to risk management and climate change adaptation, whereby there is a strategic ally for the development of these issues, linking them to the Central American road infrastructure.

It is worth saying that this Manual was jointly prepared by a Central American consultant, with technical experts from the Transport Ministers of the region and this Secretariat, in order to ensure the adequate ownership and quality of this regional tool.

Therefore, it is a pleasure to present this **"Hydrologic and Hydraulic Technical Considerations Manual for Road Infrastructure in Central America"**, so that the countries have a conceptual and methodological tool with standardized criteria, for the hydrologic and hydraulic guidelines determination and consideration in the process of planning, design, maintenance and construction of road infrastructure works, in order to help the optimal improvement of infrastructure and increase its resilience against extreme natural hydrological phenomena that occur in Central American region.

Carmen Gisela Vergara
General Secretary

HYDROLOGIC AND HYDRAULIC TECHNICAL CONSIDERATIONS MANUAL FOR ROAD INFRASTRUCTURE IN CENTRAL AMERICA

First Edition, Year 2016

Con la cooperación técnica y financiera de la Agencia de Cooperación Internacional del Japón, JICA, se ejecutó el presente documento, en acompañamiento de la DACGER y la SIECA.

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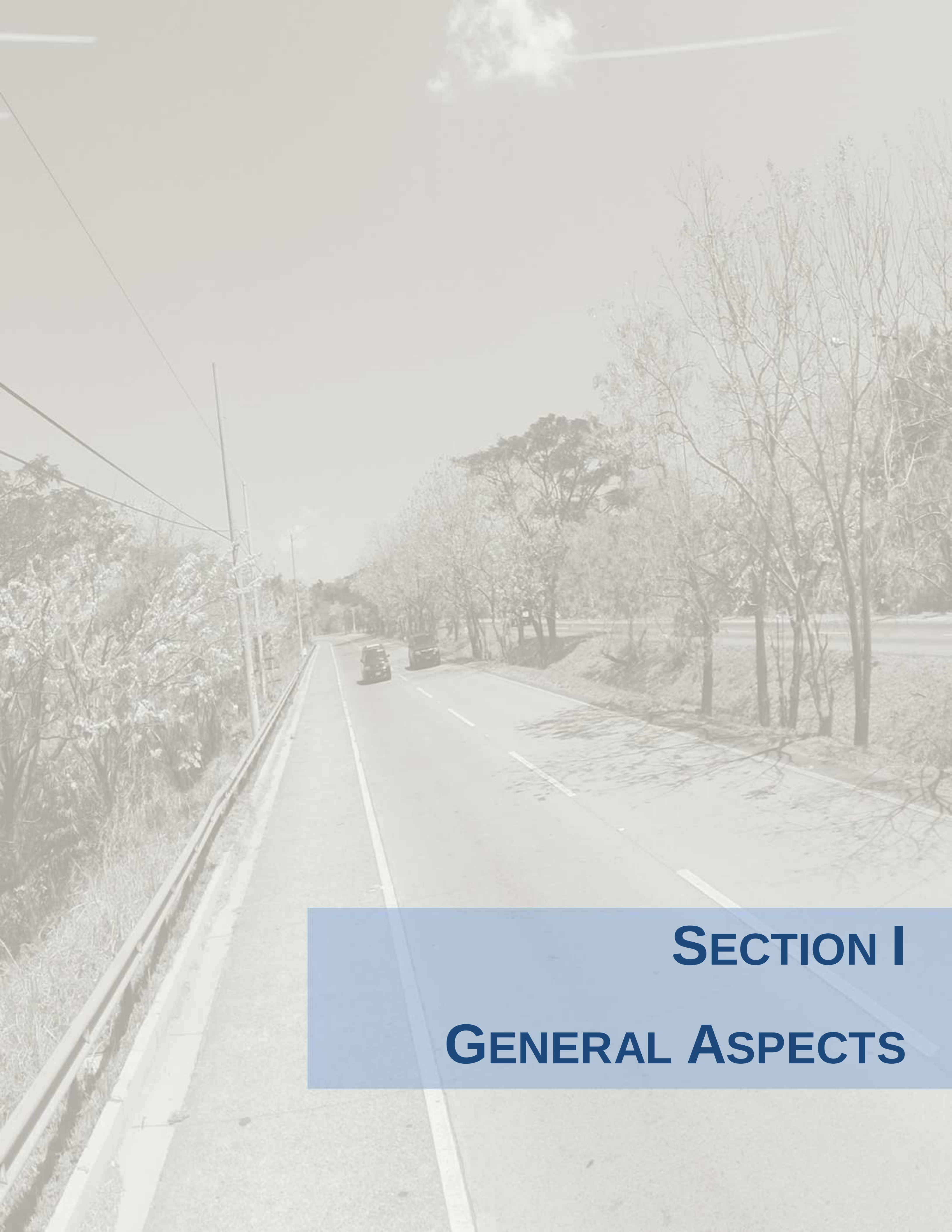


Section I

General Aspects

GENERAL ASPECTS

BACKGROUND

In 2009, with a signature of a Letter of Understanding between the Coordination Center for Prevention of Natural Disasters in Central America (CEPREDENAC) and the Central American Economic Integration Secretariat (SIECA), the willingness of both entities in supporting the implementation of the project "Regulations for Roads" was determined, in order to improve the traffic situation and reduce vulnerability of ground transportation of the region.

From this Alliance three documents were specified, which are part of the technical group that since 2000 SIECA has been working with. Such documents are:

- Update of Central American Road Maintenance Manual, focused on Risk Management and Road Security, year 2010.
- Central American Risk Management in Bridges Manual, year 2010.
- Central American Manual of Regulations for the Geometric Design of Regional Roads, focused on Risk Management and Road Security, year 2011.

Moreover, with the aim of continuing the issue in Central American region, there has been a follow-up to the agreements taken during the different reunions of the Transport Ministers Council of Central America (COMITRAN), in such meetings there were actions constructed, such as:

- Consolidation of a committee to follow-up what the Central American countries are developing in a matter of roads, with a special emphasis on what has been done by the Climate Change Adaptation and Strategic Risk Management Office (DACGER) which belongs to the Ministry of Public Works, Transportations, Housing, and Urban Development from El Salvador (MOPTVDU).
- Also, visualize synergies and joint works that maximize the topic of roads at regional level.
- Finally, invite to this process countries with valuable expertise for the technical-financial support, such as Japan.

In this sense, according to AGREEMENT No. 02-2015 of the XXXIV COMITRAN, held in Guatemala City in June 2015, it is established that:

"In the development framework of the Central American Manual of Hydraulic, Hydrology and Design for Drainage Structures in Roads from SIECA, members of COMITRAN in a period no longer than 30 day, will officially inform SIECA the designation of the national expert that will be part of the Regional Technical Committee (GTR) for the development and technical validation of such manual"

Likewise, it is agreed that DACGER from El Salvador will serve as a technical coordinator of GTR along with SIECA, and can perform the pending efforts to materialize the support manifested by the Japan International Cooperation Agency (JICA) about providing support for the development of such regional tool.

Agreed the above, the beginning is marked for the development of "Hydrologic and Hydraulic Central American Manual for the Design of Drainage Structures on Roads", that will constitute a tool that will work as a conceptual and methodologic guide for the determination of hydrologic and hydraulic parameters of design of road infrastructure drainage works.

The process of developing this manual was held in DACGER facilities, in San Salvador City, from October 12th, 2015 to February 12th, 2016.

JUSTIFICATION

Road infrastructure constitutes an essential basis for the performance of national and regional economies, at the same time; it creates a big variety of economic and social benefits. Since from its planning, the sense of increase is determined, encouraging demographic and economic development.

Many are the economic benefits generated by roads; the first and most evident one is getting in touch consumers and producers, giving consumers the opportunity to access to products of higher quantity and quality; to producers the growth of productive sectors, besides of increasing employment.

Socially, the connectivity of territory through road infrastructure moves the construction industry, which serves as a great promoter of employment generation, particularly in temporary occupations. Urban development tends to be located in the more accessible areas.

That is why any involvement on the road that hinders traffic, causes temporary disablement or worse, its complete disappearance, also affects the economic and social dynamics of the region.

Central America, a region that in recent years has proven to be vulnerable to natural disasters, mainly to extreme climate variations, its social and economic dynamics have been affected due to damage to its roads, among others.

The most emblematic example of this is recorded by the passage through the region of Hurricane Mitch in 1998, which represents one of the biggest natural disasters in recent history in Central America. This natural phenomenon directly affected the countries of the region, by partially or completely collapsing roads and bridges, whereby evidenced among other things the lack of hydraulic and hydrologic considerations in long term road designs, being a potential threat because hydro meteorological effects are becoming more intense and frequent. Some results of the involvement of this phenomenon in the countries are listed below:

Damage to road infrastructure in Nicaragua were estimated at 148 million dollars, corresponding to 1104 km of paved road, 22 bridges destroyed, 49 bridges with structural damage and 26 bridges damaged in its ramparts access. In Honduras the total damage, including direct road infrastructure, the automotive park and indirect, it was even greater and estimated at 525 million dollars. In Costa Rica it involve the attention of over 1300 km of roads due to landslides, mudslides and avalanches. More than 126 bridges were damaged and more than 1000 culverts, many of which are on the Pan-American Highway and whose damage amounted to 24 million dollars (Economic Commission for Latina America and the Caribbean - CEPAL, 1999).

Other meteorological phenomena such as tropical storm Agatha in May 2010, caused damages in Guatemalan roads. Among the adversities that caused the event, the landslides occurred in several road sections were highlighted, especially in sectors of the Pan American highways CA1 and CA-2, which is the main national and regional road axis, which serves much of the regional transit of Guatemala and international flows. The total damage exceeded 300 million dollars (Government of Guatemala, 2010)

In El Salvador, as the effects of 1513 mm of accumulated rainfall during Tropical Depression 12E were, mainly, damage to the road system with damage to structure of transport, roads section loss, more than 1,400 landslides in the highways and roads, a total of 41 bridges were damaged and 8 collapsed bridges. The total amount of damage is estimated at 223.2 million dollars, which are entirely borne by the public sector. (Ministry of Public Works, Transport, Housing and Urban Development, 2011)

Panama, although historically has rarely been affected by the hurricanes, is indeed indirectly influenced by cloudiness and, therefore, the heavy rains that are generated in its territory, causing damage to the main road at considerable cost.

From the background in the region and the uncertainty of the possible effects caused by the change in weather patterns, preventive measures should be taken to safeguard the lives of people and minimize damage to the road infrastructure and economic activities. Therefore, a good system of roads can be considered as a good tool in the region, being of great importance to entirely introduce the criteria for reducing vulnerability to the possible occurrence of new events that cause damages to this.

OBJECTIVES

- Enhance adaptive capacities of Central American territory to climate change, its current and expected effects, through the development and unification of basic hydrologic and hydraulic guidelines that contribute to making adjustments in current designs of road infrastructure.
- Understand and analyze the current situation of the Ministry of Transport of Costa Rica, El Salvador, Guatemala, Honduras, Nicaragua and Panama about the hydrologic and hydraulic provisions in designing drainage structures on roads.
- Develop a guide of elements belonging to the project environment to be included in the planning stages of road projects.
- Mention the main considerations and methodologies to consider for conducting hydrologic and hydraulic studies for the design of drainage works on roads.
- Highlighting the importance of the protection works accompanying the drainage structures on roads.
- Include in the document the particularities in the design of drainage from each of the six countries in the region.
- Develop a conceptual guide to serve as a technical tool for professional engineers in the design of drainage structures on roads.
- Complement the collection of technical manuals developed by SIECA.

SCOPE

- The manual constitutes a conceptual and methodologic guide with guidelines for the determination of hydrological and hydraulic parameters for technical considerations in the design of the drainage works on roads.
- The manual has not been developed in order to resolve conflicts between the parties involved in a project, it represents only an element guide to consider for design. If found any difference of opinion between what is stated in this document regarding legislation or national official use, the provisions of the latter should be applied.
- Part of this document is based on hydrologic and hydraulic regulations and provisions currently implemented in each country or the design of drainage works on roads, it is intended to cover the structures and methodologies mostly used in the region.
- The manual considers risk management elements to take into consideration at the stage of planning and design of roads and bridges at regional level.
- Methodologies and criteria set out in this manual are technical standards for the designer's work. Other methodologies and criteria can be used as long as it is shown by calculation reports the validity of the results.

STRUCTURE OF THE DOCUMENT

The Hydrologic and Hydraulic Considerations Manual for the Road Infrastructure in Central America has been integrated as follows:

In section 2 there is a ***diagnosis*** about the current situation in the transport ministries of Costa Rica, El Salvador, Guatemala, Honduras, Nicaragua and Panama regarding to the way the road projects are being executed: operating units, technical reference documents to define design criteria for drainage works, needs; sources of information and reference documents, potential and needs expressed by different actors consulted in each of the countries are mentioned.

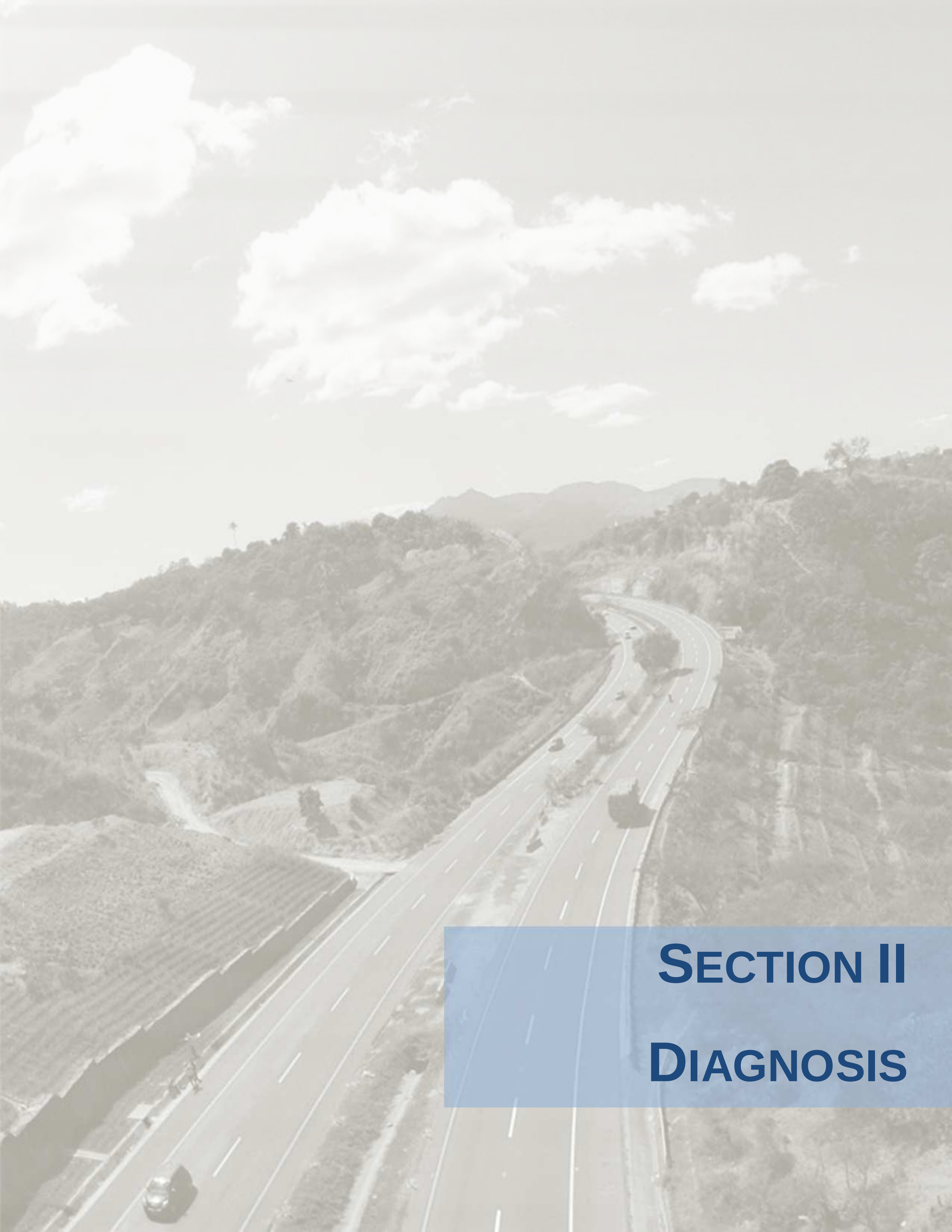
In section 3, Considerations in the planning projects stage,it is mentioned in a general manner the importance of taking into account at the planning stage of the project aspects: identifying threats and vulnerabilities of the project, the risk estimate based on site analysis and vulnerability, risk reduction measures and cost estimation, evaluation of better project alternative, analysis of specific points need for structural change due to lack of capacity of the current service (structural or hydraulic), and if necessary, the inclusion in the budget, resource analysis and construction of prevention and adaptation works for structures to be built, when the need to reconsider the work according to current conditions in the area observed.

Section 4, ***Hydrologic Considerations*** in the design of drainage works, describes statistical methodologies for the treatment and filling of missing hydrological data. Besides, some tests for data consistency, and methodologies widely used in the region for calculating the maximum flow.

Sections 5, Hydraulic Considerations in the design of drainage works, criteria and methodologies for the design of hydraulic roadworks are explained, knowing the importance in controlling the runoff of pluvial and fluvial water.

Section 6, ***Protection Works,*** mentions that the protection of a work is to implement preventive measures in order to reduce vulnerability and damage during extreme events and avoid risks to stability. That is why this section describes considerations to bear in mind the works of highway safety.

By last, in section 7, the ***Reference Sources*** and criteria for the design of drainage works on roads are mentioned, which are used in each of the countries in the region. This section is mainly based on data collected in the Ministries of Transport, as well as official documents and technical references, mainly from the AASHTO and FHWA.



SECTION II

DIAGNOSIS

DIAGNOSIS

This section is the result of consultations with the Ministries of Transport of Costa Rica, El Salvador, Guatemala, Honduras, Nicaragua and Panama, and summarizes the current status of implementation of provisions hydrologic and hydraulic design of drainage works on roads.

Among the objectives of the embodiment of this diagnosis there can be mentioned the following:

- Identify problems in the application of hydrologic and hydraulic provisions by obtaining information from primary sources, in this case, with members of the Regional Technical Committee (GTR) appointed by the Ministries of Transport of each country in the region.
- Complement the information obtained through the local GTR by means of secondary sources. This involves the collection and review of technical reference information used in each of the transport management institutions or the application of hydrologic and hydraulic design provisions for road drainage structures.
- Consult specialists and other actors involved in the field, in order to know and record the needs and contributions from a perspective outside the government institution.
- Arrange and prioritize those recorded needs of different sources of information.
- Prioritize the issues, which should focus on the **Hydrologic and Hydraulic Technical Considerations Manual for Road Infrastructure in Central America**
- Identify other potential actors who can engage in medium or long term in the continuity and improvement of the manual through the development of methodologies consistent with the reality of the region.

2.1. METHODOLOGY

As mentioned above, the main objective was to understand the current situation on the application of hydrologic and hydraulic measures in the design of drainage works in Central America, whereby a working trip was made by the transport management institutions in Central America (October 26 to November 14, 2015). The sources have been classified as follows:

2.1.1. Primary Reference Sources

The primary reference source were the member of GTR. Initially supported by the DACGER and SIECA and the use of "online" free access tool "Google Forms" it was prepared a questionnaire about the hydrologic and hydraulic design provisions for road drainage structures used in transport ministries of each country. Such questionnaire was sent to GTR members prior to working trip.

The questions can be found in the APPENDIX of this document.

The dynamics in each country was first in corroborating the answers listed in the questionnaire with members of the local GTR and then obtain information from other sources related to the topic. GTR consulted members are detailed in Table 2-1:

Table 2-1 List of primary reference sources by country

COUNTRY	GTR MEMBER	UNIT / INSTITUTIÓN
Costa Rica	Antonio Romero Castro Christian Fernández Camacho	Directorate of Bridges and Sectorial Secretary of Planning from the Ministry of Public Works and Transport (MOPT)
El Salvador	Emilio Martín Ventura Aníbal Henríquez Erick Menjívar Bernardo García Prieto	Directorate of Public Works Planning (DPOP) from the Ministry of Public Works, Transport, Housing and Urban Development from El Salvador (MOPTVDU)
Guatemala	Juan Carlos Galindo Dionisio Villegas Cansinos Víctor Vinicio Barrios	General Directorate of Roads (DGC) from the Ministry of Communications, Infrastructure and Housing
Honduras	Gustavo Ramón Suazo Hugo Fernando Martínez	Department of Hydraulic Works from the General Directorate of Public Works of the Infrastructure and Public Services Secretariat (INSEP)
Nicaragua	Jerónimo Ignacio Sánchez Joaquín Guevara Arce Fidel Rodríguez Orozco	Technical Studies Office form the Planning Department of the Ministry of Transport and Infrastructure of Nicaragua (MTI)
Panamá	Porfirio Rangel Moreno Jean Michael Guelfi Jiménez	Drainage Section of the National Directorate of Studies and Design of the Ministry of Public Works (MOP)

2.1.2. Secondary Reference Sources

Secondary sources of information were mainly made up by local technical manuals in digital format, provided by members of the GTR. In some cases, members also provided Terms of Reference examples of recent projects, in order to review some existing provisions in the design of hydraulic works.

Secondary reference sources are shown in Table 2-2:

Table 2-2 List of secondary reference sources by country

COUNTRY	MAIN REFERENCE DOCUMENT
Costa Rica	• Hydrologic and Hydraulic Design of Minor Drainages in Roads (Graduation thesis from Ramiro Gamboa, (1969); • Construction Manual for Roads, Highways and Bridges. Prepared by the Department of Regulations, Building General Directorate of the Ministry of Public Works and Transport (1983)
El Salvador	• It does not have a local reference manual. Hydrologic and hydraulic provisions are based on international manuals
Guatemala	• It does not have a local reference manual. Hydrologic and hydraulic provisions are based on international manuals
Honduras	• Roads Manual. BOOK 6: Drainages and Bridges. General Directorate of Roads (1996) • "Design and Building Procedures of Hydraulic Works Manual"
Nicaragua	• Hydraulic Guideline for the Design of Drainage Works and Rural Roads. Editions 2004 y 2011.
Panamá	• Approval Plan Manual. Executive Directorate of Studies and Design. Plan Review Department. Ministry of Public Works. (2003) • Regional Analysis of Maximum Floods of Panama. Period 1971 -2006. Company: Empresa de Transmisión Eléctrica S.A (2008).

2.1.3. Others Consulted References

Meetings were also held with other involved actors on issues of hydrology and hydraulics in the design of drainage works. It includes the Academy, private consultants, institutions related to the management hydrologic and hydrometric data, among others.

In table 2-3, the list of actors consulted is shown:

Table 2-3 List of other actors involved in the issue

COUNTRY	CONSULTED ACTORS	INSTITUTION
Costa Rica	Eyden Ajoy Arnaez	Engineering designs of roads Directorate from the Ministry of Public Works and Transport of Costa Rica.
	Esteban Cruz	Engineering designs of roads Directorate from the Ministry of Public Works and Transport of Costa Rica.
	Luis Villalobos	The National Highway Council of Costa Rica (CONAVI)
	Rafael Murillo	Hydraulics Laboratory at the University of Costa Rica (UCR)
	Jorge Granados	Department of Basic Studies of the Costa Rican Institute of Electricity of Costa Rica (ICE)
	Roy Barrantes Jiménez	Management and Evaluation Unit for Roads – PITRA. Bridges Unit–PITRA National Laboratory of Materials and Structural Models (LanammeUCR)

COUNTRY	CONSULTED ACTORS	INSTITUTION
Guatemala	Eddy Sánchez Estuardo Jerez Santos Paris Rivera Mónica Cueto Fulgencio Garavito	National Institute of Seismology, Volcanology, Meteorology and Hydrology (INSIVUMEH)
	Ligia Milithza Méndez Juan José Hanser Santos Mejía Aquino Juan Manuel Gutiérrez	Executing Unit of Road Maintenance of Guatemala (COVIAL)
	Ing. Joram Gil	Regional School of Sanitary Engineering and Water Resources (ERIS) of Guatemala
Honduras	Miguel Ángel Matute	Bridges Department of the Genera Directorate of Public Works of the Infrastructure and Public Services (INSEP)
Nicaragua	Víctor Rogelio	National Autonomous University of Nicaragua
	Néstor Javier Lanza Miguel Blanco Chávez	National University of Engineering of Nicaragua
	Juan Carlos Valle	HIDROTEC S.A Nicaragua
	Antonio Alvarado Cuadra Elmer Antonio Bervis	Private Consultants of Nicaragua
	Pedro Martínez Jáenz Fabio Guerrero Osorio	Environmental Management Unit of the MTI from Nicaragua
	Eduardo Acuña	Technical advisory consortium IDOM-NCG-METEOSIM-CONDISA for the project Technical Assistance (Short and Long Term) of Developing Climate Change Adaptation Ability in the Transport Sector of the Ministry of Transport and Infrastructure of Nicaragua.
Panamá	Pilar López Diego Arturo Jaen	Department of Hydrometeorology from Company: Empresa de Transmisión Eléctrica S.A of Panamá. (ETESA)

2.2. SOURCES RESULTS

2.2.1. Planning Stage

As a first point of the results of the visit to the different countries, is worth mentioning that the main activities of road management of the Ministries of Transport, it is centralized at its headquarters. It is not surprising that planning units or directorates have limitations addresses technical and financial resources, to meet the basic needs of inspection and consulting projects to be carried out in the future.

The implementation of a road project, in the case of a new work, usually responds to a government plan, to a specific request of the inhabitants of a community or the fulfillment of previous commitments by local or national governments. Many activities in the various institutions are aimed at restoring existing works or routine and periodic maintenance of roads.

The bias in the actions, from the planning units of the ministries of transport, is to start with a planning phase of roadwork whose result is reflected in Terms of Reference (TOR) of the project. Subsequently, the TOR are put out to tender to be executed by consulting firms. Then the function of the units is monitoring the project in its different phases. Worth mentioning that in some cases, Ministries of Transport provide technical assistance in the implementation of road projects carried out by local governments (municipalities).

During the planning stage of the road project, it is not common the involvement of other institution or body. Depending on the location and size of the project, the environmental institution in each country can intervene to grant the necessary permits for the implementation of the project. In other cases, because of the presence of aqueducts at the site of the construction place, the institution responsible for water supply or health may intervene in case of a possible contamination of a water body.

The project's impact on the environment or the work environment are executed by the project formulator and evaluated, where corresponding, by environmental institutions in each country. During the planning stage, the initial information regarding the location is the topographic maps available on ministries and secretariats. There is little reference to databases of documented historical information or data such as the basin in which is the proposed site, the presence of wetlands, water sources, dams, background on flood risk maps, geomorphological maps, geological maps, updated using soil or watershed management plans, location of weather seasons, etc. These are elements that facilitate hydrological and hydraulic studies in the design phase. Some of this information, where meriting, it is requested in the TOR to the consulting firms hired for the project design.

If there are road data bases in the ministries of transport, they are focused on the road structures as such, particularly in bridges, and there is too little record of environmental information and nor are there, within the planning units, standardized guidelines for recording and processing field data: sheets or forms to create a database.

2.2.2. Reference Technical Documents

In each country, there is more than a reference document for the establishment of hydraulic-hydrologic criteria in the design of roads. It is common to use these criteria based on American literature, mainly from the American Association of State Highway and Transportation Officials (AASHTO) and Federal Highway Administration (FHWA) for the preparation of the Terms of Reference (TOR) and for monitoring of projects.

Local technical documents, with regard to the hydrological and hydraulic part, have a similar structure in their content and are adapted from foreign manuals. In some cases, according to the technicians consulted, given the antiquity of the date of its publication, it is necessary to check the validity of the information used as a basis. For example, the Intensity - Duration - Frequency (IDF) curves used in the calculation of design flows for some methods.

The reference documents are indicated in Table 2-2.

2.2.3. Hydrologic Considerations

Usually it is asked to the hired consultant for the project implementation the basin to be defined based on the largest scale topographic maps available in the country. There are few cases in which the use of aerial photography is requested, not a criterion of scale of work is set depending on the surface basin.

It can be identified that the hydrologic and hydrometric data is a major factor and constitutes an element of hydrologic studies with major drawback presented at the time of this analysis. The data collection is usually not easy, either by the dispersion of the data in different institutions, by restricted access, for the economic cost or the lack of information. This hampers the implementation of some of the most recommended methods of hydrologic analysis and estimation of maximum flows, since much overuse of empirical methods is criticized, but little is to improve the coverage and accessibility of statistical information required by methods that are more precise. So the vicious cycle continues and at the end, it is not known whether resources are being wasted with excessive designs or increasing the vulnerability that seeks to reduce insufficient designs. Possibly, the two things are happening.

The institutions, which have hydro meteorological data available in each country of the region, are:

- COSTA RICA:
  - Costa Rican Institute of Electricity (ICE),
  - National Meteorological Institute (IMN), and
  - University of Costa Rica (UCR)

- EL SALVADOR:
  - Ministry of Environment and Natural Resources (MARN)

- GUATEMALA:
  - National Institute of Seismology, Volcanology, Meteorology and Hydrology (INSIVUMEH),
  - National Electrification Institute (INDE),
  - Ministry of Agriculture, Livestock and Nutrition (MAGA),
  - Municipal Water Company (EMPAGUA)

- HONDURAS:
  - Departament of Water and Climate Services, office of the Bureau of Water Resources the Secretariat of Natural Resources,
  - Hydrology Unit of the National Electricity Company (ENEE),
  - The National Meteorological Service, office of the Bureau of Civil Aviation and,
  - Permanent Contingency Commission (COPECO)

- NICARAGUA:
  - Nicaraguan Institute of Territorial Studies (INETER)

- PANAMA:
  - Company: Empresa de Transmisión Eléctrica S.A (ETESA)

The methods presented are directed to calculating the maximum flow. In all countries, it is commonly used methods based on the rainfall-runoff relationship, especially the rational method to calculate maximum flows.

There are differences in limiting the use of rational formula. This limitation is set as a function of surface drainage at the point of execution of the work. The variation is from 1 to 4 km^2. In some cases, it allows to use it on surfaces for up to 12km^2 and more, if compared with other method.

For areas exceeding the limit set by the rational formula, in some cases drainage analysis methodology used is left to the professional's discretion. In other cases, it is recommended to consultants, those available on the website of the FHWA. Exceptional cases are Nicaragua, Panama, and Honduras that have a fairly comprehensive guide to the methodology used.
In cases where the professional is the one who decides the methodology used, it is required a calculation log which clearly justifies the results obtained.

It should be noted that regardless of the method used for calculating flow rates, any coefficient used in the analysis, such as the runoff coefficient, it is usually taken from literature or foreign references, although there are isolated cases, as that of the University of Costa Rica, where possible variation coefficient is analyzed according to the return period for which it is estimated the design flow. For reasons that are beyond the scope of this diagnosis, it has been observed that there is little research to determine local coefficients.

2.2.4. Hydraulic Considerations

As for the hydraulic considerations, there is considerable similarity in the criteria adopted and the type of works in the projects implemented. This may be because many references from the United States are used in the requirements (AASHTO, FHWA). Variations occur in the criteria for defining a higher or a lower drainage, minimum pipe values and other reference values of the components of the structures.

2.2.5. Needs expressed by the consulted actors

According to the statement by the different actors and the documentation consulted during the visit to each of the Ministries of Transport of the region, needs arise that should be taken into account for the performance of this version of the manual and for future issues arise. Among the most important needs can be enlisted:

- Access from the ministries of transport to hydrological, hydrometric data, land use maps or data from basins, facilitating the use of more developed hydrologic methodologies for calculating the value of maximum flow and for comparing the results with those obtained through traditional methods.

- Development of hydrologic and hydraulic methodologies and runoff coefficients and roughness coefficients best suited to the natural conditions in Central America.

- Promote through SIECA the creation of a regional database of roads, where regional history can be included, as well as data to facilitate hydrological and hydraulic studies.

- Encourage regular updating of the hydrologic and hydraulic manual for the design of drainage works on roads through the development of the potential identified in each country and the involvement of new actors so that through further research, national or regional, it will contribute to the unification of criteria in the design of the works.

- Include components of risk management in the planning of road projects.

- Depth study of subsurface drainage and road design criteria and methodologies for assessing scour structures.

- Prevent the publication of regional documents conflict with local regulations or documents for official use.

- Introduce the use of statistical methods for the treatment of hydrologic and hydrometric data and methods to corroborate the validity of the results obtained.

- Always try to develop reference documents with a practical insight, aimed to technicians, where nomograms or easy use graphics are included.

- Take into account the professional profile in charge of conducting hydrologic and hydraulic studies.

- Include notions about using IT tools to perform hydrologic and hydraulic analyzes when designing drainage structures.

2.2.6. Identified potential in the region

Despite the amount of needs identified in the region, to improve the implementation of hydrologic and hydraulic provisions in the design of drainage works, there are existing capacities that constitute a potential that, with proper regional planning, can be developed and help improve future editions of the manual.

Here are some of the potential of each country:

2.2.6.1. Costa Rica

- In Costa Rica local literature related to hydrology and hydraulics is produced, in addition to other topics related to roads. At the Civil Engineering School from the UCR, information can be found in hydrological studies, estimation of maximum flow rates for design of hydraulic works. All courses are subjects of the curriculum of the degree in civil engineering and master's degree in hydraulic engineering. Moreover, they are often subject to final papers for graduation at bachelor and master degrees. ICE recently published a comparison of probabilistic and empirical methods for estimating maximum flows of small drainage basins works, such as master's thesis in hydrology from Engineer Priscilla Riggioni (2015) from UCR.

- They have the National Laboratory of Materials and Structural Models (LanammeUCR), an academic research institution attached to the Civil Engineering School at the University of Costa Rica, It is a specialized laboratory in applied research, and teaching and technology transfer in the field of protection of the civilian infrastructure, road and lifelines.

- LanammeUCR has a Transport Infrastructure Program (PITRA) whose aim is to develop and maintain transport infrastructure of the country with efficiency, quality, and safety; for the purpose of improving the quality of life and competitiveness of citizens".
- Within the Transport Infrastructure Program, the following document has been prepared: Methodology for evaluating the vulnerability of culverts by hydraulic capacity, using Geographic Information Systems (Vargas & Garro).

- Also, LanammeUCR activities cover the different areas of transportation engineering, which are addressed in a comprehensive and complementary way through seven units: Technical Audit, Development of Technical specifications and Technology Transfer, Management and Evaluation of the National Road System, Municipal Management, Materials and Pavements, Bridges and Road Safety and Transport.

2.2.6.2. El Salvador

- The Department of Climate Change Adaptation and Strategic Risk Management (DACGER) has been created as part of the Ministry of Public Works in El Salvador, specialized and totally focused on the adaptation of public infrastructure to climate change, and preventive risk management. The purpose of the unit is to develop technical and scientific studies to adapt the social and productive infrastructure of the country to climate change; as well as to design and propose mitigation works and preventive measures to reduce vulnerability and the impact of extreme events. (e.g. Basic Guidelines for Adaptation to Climate Change in the Design of Bridges in El Salvador)

- It receives technical support from Japan International Cooperation Agency (JICA) with GENSAI project, which consists of Capacity Development of DACGER for strengthening public structure. (Ministry of Public Works, Transportation, Housing and Urban Development)

2.2.6.3. Guatemala

- In Guatemala, there is the Regional School of Sanitary Engineering and Water Resources (ERIS), which operates within the School of Engineering of the University Of San Carlos Of Guatemala. Among its purposes is to provide advanced education, completing the theoretical and practical teaching, with applied research activities, according to the progress of science and technology, taking into account the needs and resources of the Central American means.

- Focuses on solving specific problems, arising from the needs of the matter, emphasizing the use of own resources and taking into account the interrelationships of sanitary engineering and hydraulic projects with fields of socio-economic development and the environment. Therefore, it is a research organization, which can make important contributions to improve design specifications or future regulations.

- Also the General Directorate of Roads of the Ministry of Communications, Infrastructure and Housing –CIV-, has an office called Technical Consultancy of River Engineering –ATIR-, responsible for planning, managing and monitoring projects aimed at proper treatment of riverbeds that have or are causing damage to the road infrastructure in charge of CIV. For this purpose, it has been promoting since 2006 the completion of all necessary studies (topography, hydrology, geomorphology, geotechnics, hydraulics, etc.) to formulate the "Treatment Plan for Riverbed", prior to the execution of works of mitigation, control and / or protection (structural measures), same ones that as well as protecting the road infrastructure, they are also aimed at restoring the natural dynamic equilibrium of these water bodies, according to the plans that promote development and sustainable management of natural resources of the country.

- It is interpreted that although exprofeso functions, objectives and scope of this office does not indicate that the project design are an adaptation of the road infrastructure to climate change and to preventive risk management, in their current concepts, the results in practice if demonstrated.

2.2.6.4. Honduras

- They have the "Design and Construction Procedures Manual of Hydraulic Works" prepared by the Engineer Yoshihiro Takemoto. This document was produced from the collection of important aspects of hydraulic engineering with field observations in Honduras.

- Mr. Takemoto developed a second consultation document entitled "Practical Measures to Prevent Flooding and Damage to River Banks, Borders and Bridges", which collected technical data of great importance to the development of control measures and important observations that result from the various work visits that he had to perform in the country and serve as a guide for the best preparation of the work undertaken and to be undertaken in the Department of Hydraulic Works.

- It should be noted that both documents were made specifically for Honduras after Hurricane Mitch in 1998, they are frequently used as a reference in the Department of Hydraulic Works from the General Directorate of Public Works of the Infrastructure and Public Secretariat (INSEP), and therefore they constitute a good line basis to provide continuity in the country.

2.2.6.5. Nicaragua

- At the time of completion of this document, the Ministry of Transport and Infrastructure of Nicaragua (MTI) is carrying out the project "Technical Assistance

(Short and Long Term) Developing Adaptive Ability to Climate Change in the Transport Sector" through a donation Nordic agreement and executed by the consortium IDOM-NCG-METEOSIM-CONDISA.

The project includes 5 components which are:

- Institutional strengthening,
- Preparation of future climate scenarios,
- Standards, design manuals, policies and legal instruments reviews,
- Pre-investment studies and
- Pilot projects.

The description of the components can be seen in the Fig.2-1:

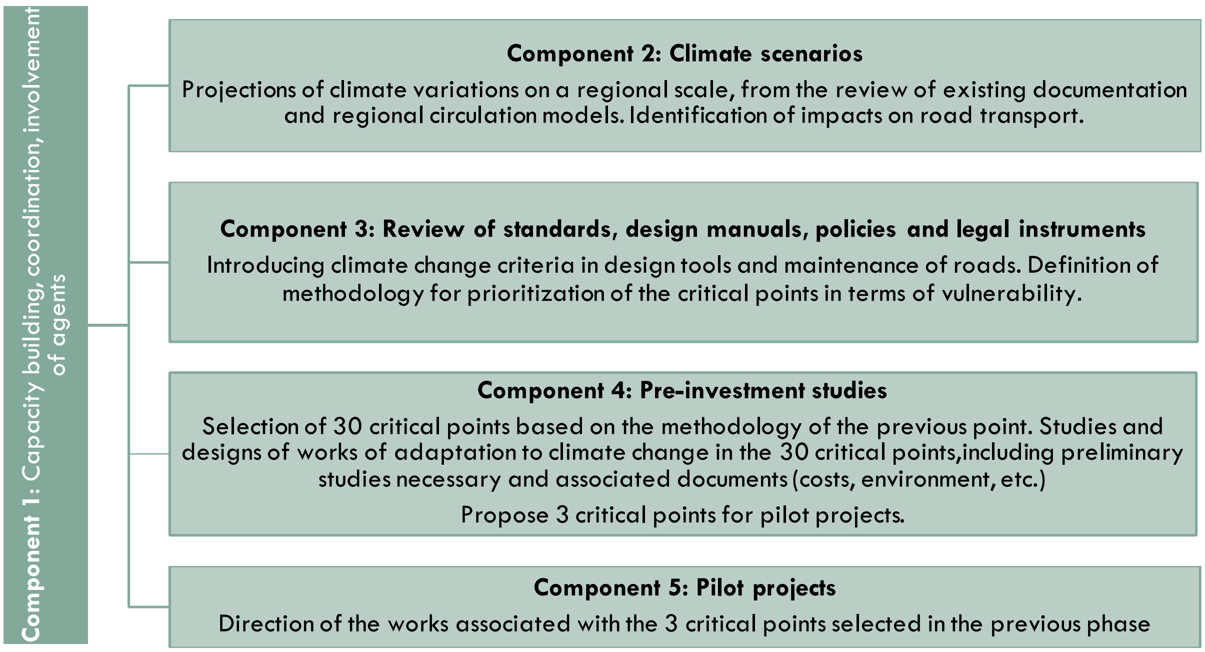


Fig. 2-1 Components from Technical Assistance (Short and Long Term) Developing Adaptive Ability to Climate Change in the Transport Sector" [Source: Engineer Eduardo Acuña Birabén, Technical Consultant of the Consortium, November, 2015]

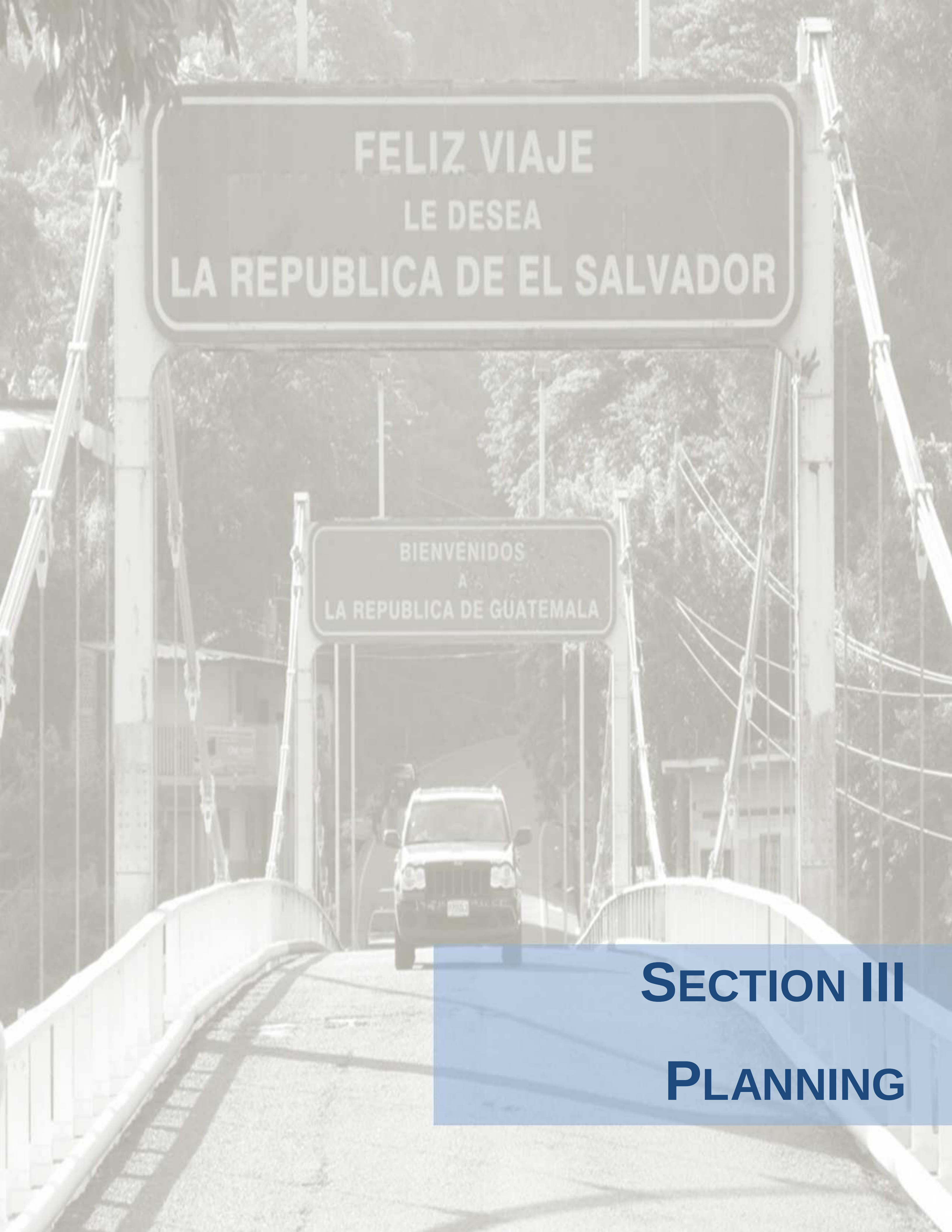
Not necessary to add the potential of the results of this assistance to Nicaragua, it can be evaluated and taken up by other countries in the region in the near future.

2.2.6.6. Panamá

- The Company: Empresa de Transmisión Eléctrica S.A from Panamá (ETESA), in charge, by law, of the hydrologic and hydrometric tool network, in 2015 it supported the performance of graduation work from students Alcely Lao and Antonio Pérez, both of the Technological University of Panama, entitled "Intensity-Duration-Frequency Relations Generations for Basins in the Republic of Panama", in which there was an update in the IDF curves. With this work, it was created a renovated tool that provides a better development of hydrologic projects in Panama.

- There is an initiative by ETESA to make available to different users, hydrologic and hydrometric data of different seasons through its website. Although there is no definite date of implementation of this measure, it may represent a major advance for both planners of road projects as well for executors of the same.

- ETESA, through its department of hydrometeorology, It is conducting the first modeling to estimate future climate scenarios. To October 2015, they are in the process of validating 40 years, from 1969 to 2009, and then be able to project in the future. The result of this work could be paid to the development of a better future hydrologic analysis.



SECTION III

PLANNING

PLANNING

3.1. GENERAL CONSIDERATIONS IN THE PLANNING STAGE AND PLACEMENT OF ROAD PROJECTS

Several activities should be carried out in the planning stages of road projects. Among the first and most important is coordination between the different technical units of the institution and planning, as well as other institutions and organizations involved in the project. This activity involves:

- The identification of the necessary permits for the implementation of work and establishment of contact with the institutions that grant them. Good relations between institutions facilitates clearance of doubt before any process.
- The exchange of information identifies the existence of other plans to run in the basin in which the work will take place, which would prevent overlapping projects at similar points.

Other activities include the identification of potential problems in the design stage, possibly caused by the presence of floodplains, presence of wetlands, water supply networks, and drainage networks or other structures and environmentally sensitive areas.

In addition, it is recommended to provide information to local residents about the works that have been announced and expected benefits, this is how the generation of rumors that might cause misunderstandings and even opposition to the project implementation, are avoided. Take into account also that the inhabitants of the villages near the project area are a good source of information on natural events occurring in the area.

The American Association of State Highway and Transportation Officials (AASTHO) suggests that in the planning stage should be taken into account the development of inventories, preparing mathematical models, forecast economic and population growth in the area of design, development and evaluation of alternative transport, the advice for the selection of the best alternative and monitoring and reassessment of the planning process, as a continuous function.

With regard to the hydraulic part, at this stage it will be necessary to perform an inventory of basins, wetlands, water supply networks, existing drainage networks, dikes or dams, bridges and historical events related to water flow. This inventory, along with other factors, will help in the development of hydraulic studies during the design stage.
Also according to AASHTO, between the planning and design stages, there is an intermediate stage between the two which is often mixed, in which the location of the road is defined in order to meet the overall objectives of the transport system as well as the local needs of the surrounding area.

In general, since it depends on the conditions of each project, the recommended steps to be followed in carrying out studies to determine the location of roads are:

- Preliminarily determine the requirements of the road: the road type, among other features.
- Select corridors or routes to follow and identify the main alternatives.

- Examine reports of planning and develop preliminary surveys to collect information on the density of the population and the growth rate, the development of land use, circulation patterns and bias, economic, social and environmental conditions that must be considered in selecting the location of road alternatives.
- Prepare the plan and preliminary profile for each alternative route, in such a way that costs can be estimated to determine the feasibility of the construction of each alternative.
- Of the alternatives evaluated, verify what powers of enforcement for further study and development of the profile.
- Perform the most comprehensive studies of the alternatives selected in the previous step.
- Prepare a final report on the best alternative location on the route, including advantages and disadvantages over the other options, as an aid to decision making.
- When the most viable alternative is selected, it proceeds to the design stage.

Participation of the hydraulic specialist during this intermediate stage should ensure adequate inclusion of the many elements that affect or are affected by drainage structure.
In projects involving drainage works, legal issues referred to in the legislation of each country must be considered. It is necessary to know the responsibility that comes with the damage caused by floods; another aspect to consider is the environmental impact of the construction of drainage and the work itself.

Therefore, in the planning stage and location of the project, it is important to consider that the course of streams is divided into three zones or main parts (sections of the river), according to its erosive capacity and sediment transport: the steep gradient (or mountain) curves, or upper reaches the steep gradient (or mountain) curves, or upper reaches -, the average gradient (or piedmont or mountainside) -or mid reaches- and low-gradient (or floodplain) –or low or inferior reaches -. Therefore, a road can be located in any of the three main parts or areas and flow characteristics that cross the road must be considered to determine possible changes on the drainage pattern. If as a result from these considerations, it is required to make major changes to the structure, more comprehensive and detailed studies should be developed regardless of whether the project is at an early stage.

Moreover, some problems may arise in the design stage because of possible effects that were underestimated in the planning stage. This is because many times in the early stages there is no field information or detailed information. Therefore, any recommendations resulting from the initial stages should not be accepted as a final solution, nor any binding commitment must be purchased.

Potential problems that may be taken during the construction of the works shoul be considered; and the use of temporary structures. Examples of some problems in the construction phase are:

- The occurrence of erosion and sedimentation. The appropriate risk control measures must be previously considered.
- Climate variations during the year, which implies the need to build certain elements of the previous structure to a rainy season, which can prevent flooding in the area.

Finally, the effects of road construction on the environment should be taken into account in the planning stage. This includes effects on water quality in aquatic fauna, landscape, etc. This information is usually included in the Environmental Impact Study that accompanies each project.

3.2. PROJECT LOCATION SITE CONSIDERATIONS OR SITE ANALYSIS

3.2.1. Fluvial Geomorphology

During the design of the drainage works on a road is necessary to study the river, how it is formed, how it behaves to natural changes and human actions, and how they behave without external influences. In this sense, fluvial geomorphology, geomorphology specialized branch that deals with the study of landforms, rams and relief caused by the action of the streams on the earth's surface. By their field of study, this branch is often linked to hydrography.

The hydrological regime of rivers is determined by the characteristics of the basin and rainfall (Martín Vide, 2002). In tropical climates, such as in Central America, it is a remarkable strong seasonality of the hydrological regime, having periods of high water and other low water, if there is sufficient groundwater discharge, or can be dried. From this, it emerges a basic classification of rivers, calling perennial those that maintain a permanent and ephemeral flow, to those that carry water in heavy rainfall events.

Also, known as alluvial rivers, those running on generally granular, loose and deposited by the same river sedimentary materials. Such materials can have a large horizontal extension, adjacent to the rivers, forming floodplains subject to occasional periods. These plains have a changing nature so must be examined carefully to determine how they can affect whether or not the crossing of a road, as it is common in the region that many of the activities take place in this type of land.

Furthermore, in rocky bed rivers they are often fitted between valleys and are generally of a more stable nature, but should always be examined.

According to AASHTO, a classification typically used by engineers and planners of road, is according to top plan geometry that adopts the stream.

3.2.1.1. Braided Streams or with anastomosis

A braided stream is very wide and consists of multiple, interconnected smaller streams forming islands. Braided systems reflect the dynamic relationship between sediment transport, nature of the materials in the floodplain, and seasonal variations in the discharge of water flow. They cause serious problems because of its unstable nature, rapid lineup changes, degradation and aggradation, and large amounts of transported and deposited sediment. Such systems should be avoided whenever possible; it is advisable to consider other potential sites where possible.

If an intersection on a braided stream cannot be avoided, there are certain design measures that must be considered. These include: bridging on the entire width, stabilize the soil around the bridge piers and implement measures to prevent the undermining.

It is important to minimize any effect generated by the sediment transport capacity of the flow. This could cause potential changes in the river upstream or downstream of the road. Fig. 3-1.

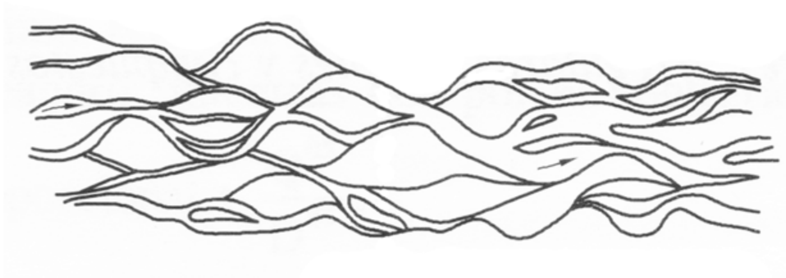


Fig. 3-1 Braided stream pattern in river (AASHTO, 2006)

3.2.1.2. Straight Streams

A second group are the streams at straight riverbeds that in the reality, creeks or streams are not really straight; even if the banks are parallel to each other, the flow in its trajectory in the deep end, usually ranges from one runway to another.

Natural straight stretches of alluvial streams are often only a temporary condition or a transient state until the time in which a meander is formed in the area. Aerial photographs, maps or field research may reveal ancient riverbeds and provide an indicator of future directions of movement. Even artificial riverbeds, often designed and constructed so straight, can be unstable. Unless they contain structures to reduce water velocity, they have steeper gradients causing higher flow velocities, which often increase degradation and aggrading upstream and downstream. Fig. 3-2.

Two situations may arise:

- First, if the road crosses a straight segment, attention should be paid to the stability of this.
- If a riverbed has to be modified to make it a straight section in order to better accommodate the road, the effects of this amendment, both upstream and downstream should be evaluated.

Fig. 3-2 Straight channel pattern in river (AASHTO, 2006)

3.2.1.3. Meandering Streams

Finally, the winding or meandering streams, the most common description is that it forms an S-shaped path with very pronounced sinuosity. They arise particularly in alluvial plains with gentle gradients. And its origin is that in the section of the riverbed where water velocity is greater, erosion increases due to centrifugal force, water digs the shore and makes a concave shape. While on the opposite side where the flow velocity is lower, sediment deposition grows and the wall becomes convex.

Road projects located near a meandering stream generally require protection from the banks to control lateral erosion. However, in some cases this involves channeling rectilinearly the riverbed resulting in the problems described in straight streams. Even if these can be treated, there is still the possibility that a meander can move from the protected zone into an unexpected direction and bypass the shield of the river canal.

In planning or locating a road project or a bridge into a stream of meanders, these potential changes of the river must be considered and the effects on the work must be evaluated. Any other work along with the stabilization of the river canal must be made up to a distance beyond which the bend cannot continue. This should be done to both, upstream and downstream of the project.

Finally, it is necessary to establish the importance of studying the drainage pattern that overlooks the river and the impact of the road on the same. Therefore, the design should not alter this pattern, when the road crosses it. Fig. 3-3.

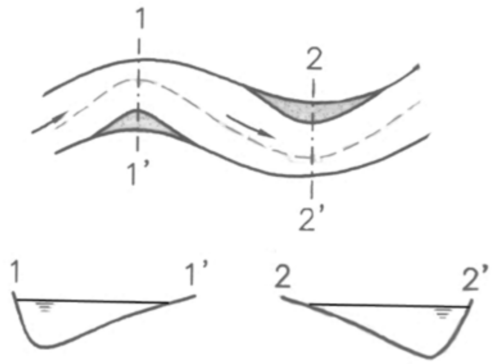


Fig. 3-3 *Meandering stream in river* (AASHTO, 2006)

3.2.2. Road Alignment

The horizontal alignment of a road determines where the crossings occur with watercourses. Two aspects of the proposed alignment should be considered:

- First, how streams or drainage systems of pluvial water can affect the road and,
- Second, how the road may affect the flow characteristics of such streams or systems.

Small changes in the alignment of the road can sometimes alter the flow characteristics. Changes that can be made in horizontal alignment will depend on whether the project is an improvement of an existing road or building a new one. However, it will be necessary to revise the alignment to identify sites where gradients must be protected against erosion, protection of the drainage and the situation of meandering in the event of its presence, or any other factor, which threatens structures.

In the case of a new road project, this allows to include hydraulic considerations from the outset to determine the alignment of this. Changes can be recommended to locate the road away from a stream or put a bridge on a more stable river canal. These recommendations should be done early in the development of a project to avoid delays during the design stage, or prior to the acquisition of right of way; after which the horizontal alignment will be difficult to change.

Also the vertical alignment should be considered to verify the influence of drainage works. Generally, it is more likely to alter and may be modified in subsequent stages of the project.

Vertical alignment or profile of the road, along with hydraulic opening, determines when and where the road will be overwhelmed by a stream. By changing the alignment, either up or down, the frequency of overflow can be increased or decreased. The profile not only affects the frequency of overflow, but also determines the flood level upstream of the area identified vulnerable of the road.

It may happen that some roads act as interceptors of the flow so that it may be required that surface runoff upstream settles into pluvial drains or diversion canals.

Flat areas can intercept overland flow although they require special drainage treatment. These issues are of particular concern with large urban highways and deserve careful consideration to locate the works.

In rivers where there is sailing, sailing ships may become the factor that controls the vertical alignment. So special care must be taken in the presence of vertical curves at intersections of these courses.

3.2.3. Others factors to consider

It is important to check the location of the road when crossing a stream or watercourse, for various reasons, including:

- It is not the same to place the road in the crossing near the confluence of two streams compared to when the flow is unique.
- High backwaters, as far as possible, should not be tolerated in any area.
- The areas with tidal influence have a completely different list of hydraulic considerations.
- The location of the crossing determines the type of drainage structure to be built. And if the structure is a bridge or culvert, it might be a difference in the hydraulic analysis.

Some considerations to take into account for the location of the drainage works on roads are:

a) Physical Characteristics of the site

A road crossing nears the confluence of several rivers or streams present a different set of considerations than when crossing an area with impact of the tide.

First, a highway junction near an area of confluence of rivers should be avoided because the hydrologic design should consider various combinations of hydrological events and their effects on the basin. In addition, flow peaks may occur simultaneously at the confluence of the rivers and make the stream less stable especially in smaller basins. Implying that the alignment of the drainage structure should be carefully analyzed.

In the case of the presence of tides, special considerations must be made in the design; taking into account changes in the water level due to these; and due to tectonic movements of the terrestrial crust, tsunamis, storms, stacking or removal of water or wind blowing from or in direction to the coast, water heating due to phenomena like El Niño and global warming.

Also, areas of marsh or wetland ecosystems are environmentally sensitive areas due to the existing biodiversity. If developed a road project in these areas, it has to be noted that the flow should not be altered or worse, restricted. These potential problems should be identified at the planning stage.

b) Land Use

The use that is given to the nearest land to the watercourse should be considered in the planning stage.

In rural areas, the construction of a drainage work on a road junction can affect private property, both upstream and downstream. Upstream, it must be considered backwater effects and increased water level during a flood because it could affect the present and future land use. It even brings losses in current crops.

Downstream, it has to be taken into account the possible increase in the flow rate passing through the drainage structure. The increase in velocity increases the possibility of occurrence of local scour and downstream aggradation of the riverbed. The water effects of the structure below are more difficult to quantify.

In urban areas, the consequences of increasing flow rates or increased level during a flood are more important to consider, since that damage to the structures installed may be higher, affecting the operation of the city and even threaten the lives of people. It will be important in these areas to have a regulation governing the limits on changes that can be made on the flow characteristics or basin.

c) Types of Structure

The location of a crossing river or stream also limits the type of structure that can be used. Depending on the site, some alternatives in form and types of structures can be recommended. Although, we must remember that in this type of decision also enter other factors such as environmental, construction and maintenance, and most importantly, economic.

3.3. START-UP DATA

In the planning stage, data collection should be as complete as possible and avoid the waste of resources due to repeated inspections to the area. It should be taken into account the information on the location of the land and magnitude of the work to be built; so it will be important previous work in office and coordination with all actors involved to carry out this activity in the best way possible.

With regard to the hydraulic and hydrologic part, the type of information will require the following:

3.3.1. Topographic Information

It is important to collect topographical information in places where hydraulic analysis is required. This type of information is useful to analyze the flow conditions in natural conditions and possible changes that may be generated by the implementation of the drainage works. Also, to identify important structures in the surrounding area, residential areas, schools, shopping centers, other roads or bridges that may be affected by the new structure.

In general, recent topographic information is difficult to obtain in the early stages of the project; map sheets, aerial photographs, digital elevation models or other documents of roads built in the area will be needed to be used. With the information gathered, there will be an idea of the information that will be updated during the design stage.

3.3.2. Characteristics of the riverbed

For detailed hydraulic analysis, it is necessary to have information on the flow profile, the horizontal alignment of the canal and the cross sections of this.

It is common to not have this type of information in the planning stages of the project. So previous analysis may be made based on topographic maps, aerial photographs or other documents or reports made to the existing river riverbed.

In addition, a method that can be very useful is taking pictures (local and aerial) to document existing conditions. This method helps to identify the components of the canal, the main material, and the type of vegetation and the presence of drag material. Photographs must be taken both upstream and downstream of the site of the drainage structure.

During this stage of the project, the required field studies for the design stage are determined. In addition, the study limits upstream and downstream of the site where the project is located, the number of cross sections required and the distance between them will be defined. The number of required cross sections may vary from the conditions of the river canal and project requirements.

3.3.3. Hydrologic Data

In addition to the physical characteristics of the area and the riverbed in the planning stage, it is also necessary to collect data of magnitude and frequency of the processes that contribute to flooding of the study area. The data can include climatic characteristics, characteristics of the basin, surface runoff, hydrometric data, watermarks and behavior of existing structures during past events. It is important for the specialist in hydrology conducts interviews with the locals, in order to learn historical data of river behavior.

a) Characteristics of the Basins

Physiographic characteristics of the basin, sub-basin or watershed study are needed in order to predict flood flows. Many of these characteristics can be determined from office studies, although some may require a field inspection. On size and configuration of hydrographic basins, the geometry of the network of rivers, the sewage volumes of ponds, lakes, reservoirs and flood plains, and general geology and lands of the basin can be found from mapping. The land use and vegetation cover may appear on maps and aerial photos; but it will have to be careful with changing land use, so the best option is to inspect the area and conduct surveys.

With these features, at times of runoff, infiltration values, storing values, and runoff coefficients can be estimated for use in calculation of magnitudes of flooding.

b) Rainfall

Other hydrologic information required are rainfall records of rainfall seasons in the vicinity of the study site. Although the use of rainfall records of seasons located outside the basin is not ruled out. The largest amount of data available should be implemented and should determine the reliability of the results obtained.

Intensity maps of the rain and weather summaries can also be used for different regions of the country, if available.

c) Data form previous floods

At sites where available, data collection related to floods should be obtained. These data includes records of measuring instruments in the river and even news stories of past events. During the field inspection, it will be important to conduct interviews with local residents, visit agencies or offices that have photos of past events, identify watermarks on structures or on the ground to be associated with the occurrence of hydrologic events.

It is important to mention that presence of small debris on the ground can also be a watermark, which should be recorded.

Finally, keep in mind that the size, location and condition of existing bridges and culverts on the stream under consideration, as it can be a valuable indicator to select the size of a new structure.

3.4. RISK MANAGEMENT IN THE PLANNING OF ROAD INFRASTRUCTURE

Hydrologic and hydraulic considerations and mainstreaming a perspective on risk reduction of natural disasters should be considered in the planning stage for road projects, bearing in mind that road infrastructure should be planned so that major changes should not be required once the works are already built.

Central America, being a developing region and to the vulnerable time before the increasingly recurrent natural phenomena, has little room for mistake to invest its resources for public infrastructure. Thus, the stage of project planning is vital and where the possible effects of the construction of infrastructure in the current conditions of the environment should be assessed.

The aim of this subsection is to provide an introductory guide to general guidelines to consider in the incorporation of a vision of disaster risk reduction in the occurrence of natural hazards. The ministries of transport or local governments will be the ones interested in the implementation of road works, and responsible for demanding the implementing of such vision in the project life cycle.

Currently, in the region, there is being used and incorporated this component in road projects; derived from the implementation of government policies in different countries. However, it still lacks strength, probably due to a lack of adequate financing for the management of risks in different countries, the absence or lack of information necessary and available for evaluation, poor communication between technical and scientific institutions that evaluate and produce information about natural hazards, and vulnerabilities with the ministries of transport, which forces to generate new or processed information from other sources to meet this analysis.

After viewing a feasible solution with its own scope to the problems identified in a region, along with the subsequent development of the project profile, he prefeasibility will constitute the stage, in the project life cycle, in which preliminary designs leading to the different technical alternatives that can meet the identified needs and the technical requirements of each alternative, will be developed. Later will come the feasibility stage where, given the technical alternative selected by its respective requirements, technical, financial, economic and social outcomes of the investment will be estimated and determined to the project formulation (Yépez, 2011).

It is in these early stages, where the need for risk analysis and its scope is defined, which means, in the case of the project done, define the location and area of influence of the structure, identify threats to which it is exposed and prioritize those that will be analyzed according to technical criteria.

At first, it is not necessary to perform detailed studies of the area of construction site during these stages. The scope of these studies will be conditioned by:

- The importance of the work and the impact on the environment if it is affected.
- The economic cost of conducting such studies
- The availability of starting information and trained technical personnel.

- The type of natural hazard that can affect the project site area.

The term "natural" excludes those threats caused by man, but it is necessary to consider that our own activities and interference in natural processes make us increasingly vulnerable, that is why when planning a road work, it is necessary to consider the responsibility in case of impact a natural watercourse or cause flooding or other possible effects on that can be incurred, like backwaters caused by restricted flow due to the construction of arch culverts or bridges, changes in the velocity of the watercourse that may affect other downstream areas, increasing or decreasing flow rates, quality degradation or alteration of subsurface water flow.

The development of a risk assessment involves different actors, such as consulting firms, academic and technic-scientific institutes, emergency committees and other ministries in constant coordination with the institution entrusted to the analysis, which will validate the results and make decisions based on these.

3.5. RISK ANALYSIS

Risk analysis begins with the definition of the objectives of this and of the performance indicators of the structure; the planning authority of the project does this. The starting information to determine these factors results from the revision of the regulations or codes for design or considerations on the behavior of the structure to threats. The accomplishment of any rule or code implicitly represents a base level performance.

In the absence of such regulations to define the objectives and performance indicators, contributions from professional groups and experts together with the adoption of regulations or documents from other countries, are a good option to consider for this definition.

3.5.1. Identification of hazards in the planning stage

After defining the objectives of the analysis and performance indicators, it should be analyzed if the structure will be affected by one or more natural hazards over its lifetime. For this, it will be necessary the collection of existing information that is related to the following topics:

- History of natural events in the area where it is intended to perform the work or in the area of influence.
- Technical reports, news stories and other material related to disasters, damages recorded in the area or on the existing infrastructure and renovations or repairs made. In addition, studies of post-disaster socio-economic impact constitute a good reference.
- Threat and vulnerability assessments previously performed near the project, conducted by technical-scientific institutions, international bodies or NGOs.
- Risk maps and risk assessments previously conducted in the area. Included in this point, topographic maps, aerial photographs and / or multi-temporal satellite images and hazard maps.
- Other documents allowing an overall view of the characteristics of the area, such as lessons learned reports and reports drawn up by bodies or committees of emergency care, to name a few.

- Interviews with residents coming to the area of the project or community associations.

It is important at this stage to work with only information available that usually is scattered in different institutions. In section 7 of this document, a reference is made on some public institutions that can guide in the search for this type of information in the region.

After gathering the information, it will be necessary to characterize the threat. One way of performing this process is by identifying representative parameters. Some of them are illustrated in Table 3-1:

Table 3-1 Representative parameters for the characterization of threats (UNASUR, 2014)

THREAT	CHARACTERIZATION PARAMETERS
Seismicity	• Ritcher Magnitude Scale (Ms) • Moment Magnitude (Mw) • Ground acceleration (g) • Intensity defined by the Modified Marcella scale (MM)
Mass movement	• Slope Gradient • Volume of Material • Reptation velocity
Tsunami	• Maximum height of flooding • Arrival time to the coast • Direction of propagation of the tide • Affected area • Return period • Extent of tsunami
Wind	• Sustained wind velocity • In the case of hurricanes, magnitude, according to the Saffir-Simpson scale.
Flooding	• Stream • Depth / height of water • Water velocity • Duration of flooding • Flooded area • Accumulated rainfall • Return period
Volcanic activity	• Type of volcano • Frequency of occurrence of the eruption • Areas exposed to lava flows, lahars, pyroclastic, ash • Duration of the eruptive process

It is important to note that the lack of information does not imply the absence of a threat, and therefore risk.

It can also happen that the scale or resolution of the information is not adequate for the project to be carried out, sin embargo, the important thing is to have an overview of the characteristics of the area in which the project aims to perform.

3.5.2. Identification of vulnerabilites of structures before the identified threats

Infrastructure consists of different components. In the case of roads, as being works that extend throughout the territory, increased attention in its analysis will be necessary because the components may be subject to different threats. In addition, there might be technical and / or financial constraints for risk analysis throughout the infrastructure. So, a prioritization of the components to be analyzed must be made according to its relative importance and degree of exposure in relation to the threat. Critical or vital components to the functioning of the structure will be part of the risk study.

Preliminarily, should be examined the possible effects caused by each threat independently on the structure or the selected component and ask whether it is able to resist. Also, set the level and type of threat to which the work will not present any damage or any technical and economic repairable damage, without collapse, and what protective measures should be implemented.

In the case of existing works, useful information for this characterization may be the calculation reports, design and construction criteria, location general plans and constructive work information, memoir or reports of the functioning of the work and reports of maintenance and damage. In addition to inspection or field surveys to ascertain the state of the infrastructure and its operation, validate information or technical specifications drawings or supplement missing information.

In the case of new works, an alternative is to compile information on experiences in the operation of similar components in other structures against certain threats.

In the end, those components considered of great importance that are located in hazardous areas will be part of the risk study.

3.5.3. Types of risk analysis

After characterizing the threats and the components of the work, it is necessary to define the scope and type of risk analysis to be performed. On those elements that are defined as very important because of the impact on the environment, if affected, it will have to be considered developing detailed studies to define the risk to which they are subjected.

The consulted reference methodology for disaster risk management in infrastructure, proposes three levels of risk analysis that can be implemented, which are described in Table 3-2.

Table 3-2 Scope of risk analysis (UNASUR, 2014)

LEVEL	TYPE OF RISK ANALYSIS	DESCRIPTION
1	Qualitative	Designed to deliver a simplified estimation of threat, vulnerability and performance of the structure analyzed. This level of analysis can be performed in a short time by technical personnel with knowledge of the type of component under analysis.
2	Deterministic	It is defined by a quantitative analysis, based on historical information and statistics to characterize threat, vulnerability and component performance, including validation and obtaining information to ground

LEVEL	TYPE OF RISK ANALYSIS	DESCRIPTION
		level. This level of analysis can be developed by technical personnel with expertise in the type of infrastructure under study with technical assistance and participation of specialists in the characterization of threats and system modeling.
3	Probabilistic	It provides detailed results quantitatively, which are based on accurate information and tools for modeling and probabilistic analysis appropriate to the state of the art knowledge. The use of advanced analytical methodologies is expected at this level, so the participation of experts and specialists is needed. In addition, this level of analysis requires significant fieldwork, laboratory testing, and instrumentation and information generation.

As mentioned above, the scope and type of analysis will be subject to the importance of the work, the time, the available information and the technical and financial resources.

Subsequently, an analysis of the performance of the structure should be carried out in order to identify risk reduction measures, either by adopting measures of prevention or mitigation. Always bearing in mind that the results must meet the objectives and performance indicators defined in the initial phase.

The performance analysis includes a review of the functioning of the structure during normal situations, emergencies, and, if possible, including a history of past disasters to identify the essential components and critical issues that may arise. This will allow estimating an "acceptable risk" according to performance indicators defined early in the process. This type of analysis will require expert judgment and the possibility of the definition of possible scenarios or probabilistic analysis.

From the result above, the possible risk, reduction measures are identified, whether prevention or mitigation that will ensure the level of performance of the structure previously established. Also, for any mitigation measures a cost-benefit analysis should be performed.
Making final decision will depend on the option of reducing identified risk, the level of performance of the estimated structure, cost and cost-benefit analysis of the option.

3.5.4. Comprehensive risk management and its links to the hydrologic and hydraulic analysis in road infrastructure

The disaster risk management is defined as processes for designing, implementing and evaluating strategies, policies and measures to enhance the understanding of disaster risk, encourage reduction and risk transfer, and promote continuous improvement in the preparedness, response and disaster recovery practices, with the explicit purpose of increasing human security, welfare, quality of life and sustainable development (Interamerican Development Bank, 2012).

People, goods and services are likely to be affected by water in any form. Hydrologic events are relevant to disaster risk if:

- Adversely affect the means of life, affecting economic and social activity of the population.
- They have consequences for the safety of people.

Hence the importance of proper hydrologic and hydraulic analysis for the design of the works on infrastructure. In hydraulic works and drainage structures, the return period flood is related to the level of risk. The higher the return period, the lower the risk.

Although accurate quantitation of the factors that make up the concept of risk can be difficult, once the risk analysis and identification of actions to reduce them is made, decisions must be taken for management based on a legal framework or cost-benefit analysis. The amount and type of strategy to follow may vary according to specific conditions of projects and available resources. Some of the standard are shown in Table. 3-3:

Table 3-3 Strategy for disaster risk management (UNASUR, 2014)

STRATEGY	MEANS	ACTIVITIES
Risk reduction	Works and actions of prevention and / or mitigation, enhancement, relocation	• Design and execution of works. • Operation and maintenance protocols
Emergency preparedness	Emergency plans, contingency, simulations, simulacra, among others	• Develop plans for personnel training • Simulacra and simulations
Risk transfer	Insurances	• Estimate the maximum probable loss • Acquire Insurance / Policy

Risk reduction can be implemented prospectively in the planning of new structures, but also in a corrective manner with existing structures. It is perhaps the main strategy to substantially reduce the vulnerability of structures to the succession of disasters.

On the other hand, preparations for the answer are strategies that are implemented considering that will be difficult to reach the point of zero risk when planning the structure; there is always a chance of it being affected by any threats. The key to this strategy is to identify weak points and critics of the structure, resulting from the risk analysis, in order to prepare resources and actions to be used during a specific event.

Finally, keep in mind that from a financial point of view, the risk of natural disasters implies a potential economic loss and the implementation of measures to ensure financial resources to partially or completely cover such losses. Risk transfer is one of the strategies in which the insurance industry, involves securitization or other financial schemes that can be integrated into the risk management in order to transfer it and is not capable of being taken by the affected.

A final strategy, perhaps more difficult to implement because economic solvency is required, is to absorb the risk. In this case, the planning institutions become their own insurers, making a regular savings to cover any emergency. The savings should be explicitly to address the repair and / or reconstruction of the damaged site, so that funds cannot be used for any other purpose.

3.6. MINIMUN REQUIREMENTS TO CONSIDER IN GATHERING INFORMATION

As a reference at obtaining information, shown in Fig. 3-4 as an example developed by DACGER for evaluating vulnerability Bridges. Each country may develop its own worksheet, according to their national requirements.

Ministerio de Obras Públicas, Transporte, Vivienda y Desarrollo Urbano — EL SALVADOR

DIRECCIÓN DE ADAPTACIÓN AL CAMBIO CLIMATICO Y GESTIÓN ESTRATÉGICA DEL RIESGO

SUBDIRECCIÓN DE PUENTES Y OBRAS DE PASO (SPOP)

DACGER GENSAI

Hoja de Evaluación de Vulnerabilidad en Puentes

DATOS GENERALES

ID DE PUENTE: ____ EVALUADOR: ____

PERSONAL DE APOYO: ____

FECHA: ____

NOMBRE: ____

RUTA: ____ TRAMO DE: ____ GPS LATITUD: ____

DEPARTAMENTO: ____ TRAMO HASTA: ____ GPS LONGITUD: ____

Factores para la probabilidad anual de un desastre (años)	Categorías de los factores de probabilidad anual de un desastre					Puntuación del Período de Retorno: PPR [año]	
Condiciones de entorno del río (vista en planta)							
Geomorfología general Puntuación del Periodo de Retorno [año]	1. Plano aluvial 5		2. Abanico aluvial 5	3. Llano de valle 10	4. Area montañosa 0	PPR1	0
Cambios históricos del curso del río Puntuación del Periodo de Retorno [año]	1. Fuera de la longitud del puente 5		2. Adentro de la longitud del puente 0		3. No aplicable 10	PPR2	0
Porciones estrechas en curso del río Puntuación del Periodo de Retorno [año]	1. Natural 5		2. Por estructuras artificiales 0		3. No aplicable 10	PPR3	0
Longitud de puente más corta que el ancho del río Puntuación del Periodo de Retorno [año]	1. Aplicable 0			2. No aplicable 10		PPR4	0
Posición de las fundaciones del puente en el río Puntuación del Periodo de Retorno [año]	1. Sección en curva (meandro) 6	2. Sección en tramo recto 10	3. Hacia aguas arriba, canal localizado dentro de 10 m de distancia hacia seccion en curva sin revestimiento 20	4. Hacia aguas arriba, canal localizado dentro de 10 m de distancia hacia sección en tramo recto sin revestimiento. 25	5. Hacia aguas arriba, en el canal con revestimiento 35	PPR5	0
Ángulo de cruce (AC) de las fundaciones del puente respecto a la dirección del flujo Puntuación del Periodo de Retorno [año]	1. AC $\geq$ 20° 0		2. 20° > AC $\geq$ 10° 5		3. 10° > AC 10	PPR6	0
Protección de la margen derecha del río por obras de revestimiento Puntuación del Periodo de Retorno [año]	1. Revestimiento con deformación severa 10	2. Revestimiento con deformación menor 15	3. Revestimiento sin deformación general 25	4. Sin revestimiento con erosión severa 0	5. Sin revestimiento con erosión menor 5	PPR7	0
Protección de la margen izquierda del río por obras de revestimiento Puntuación del Periodo de Retorno [año]	1. Revestimiento con deformación severa 10	2. Revestimiento con deformación menor 15	3. Revestimiento sin deformación general 25	4. Sin revestimiento con erosión severa 0	5. Sin revestimiento con erosión menor 5	PPR8	0

Fig. 3-4 Vulnerability Assessment sheet on Bridges

Ministerio de Obras Públicas, Transporte, Vivienda y Desarrollo Urbano
EL SALVADOR

DIRECCIÓN DE ADAPTACIÓN AL CAMBIO CLIMATICO Y GESTIÓN ESTRATÉGICA DEL RIESGO

SUBDIRECCIÓN DE PUENTES Y OBRAS DE PASO (SPOP)

DACGER GENSAI

Hoja de Evaluación de Vulnerabilidad en Puentes

DATOS GENERALES

ID DE PUENTE:

EVALUADOR:

PERSONAL DE APOYO:

FECHA:

NOMBRE:

RUTA:

DEPARTAMENTO:

TRAMO DE:

TRAMO HASTA:

GPS LATITUD:

GPS LONGITUD:

Factores para la probabilidad anual de un desastre (años)	Categorías de los factores de probabilidad anual de un desastre						Puntuación del Período de Retorno: PPR [año]	
Condiciones de entorno del río (vista en sección transversal)								
Tipo de fundación en contacto con estrato base	1. Pozo de Cimentación	3. Grupo de pilotes	3. Zapata aislada		4. Desconocida		PPR9	0
Puntuación del Periodo de Retorno [año]	10	10	0		0			
Fundación cimentada dentro de estrato rocoso	1. Aplicable			2. Desconocido, o no aplicable			PPR10	0
Puntuación del Periodo de Retorno [año]	20			0				
Relación de Profundidad (RP: Profundidad de desplante / ancho o diámetro de la fundacion)	1. Desconocido o RP < 0.1	2. 0.1 ≤ RP < 0.5	3. 0.5 ≤ RP < 1.0	4. 1.0 ≤ RP < 1.5	5. 1.5 ≤ RP		PPR11	0
Puntuación del Periodo de Retorno [año]	0	5	10	15	20			
Protección contra erosion/socavación en el cauce del río	1. No existente y la altura de caida es mas de 1 metro	2. No existente y la altura de caida es menor de 1 metro	3. Obras de protección, parcialmente en el ancho del río	4. Obras de protección en todo el ancho del río y unicamente hacia aguas arriba	5. Obras de protección en todo el ancho del río y unicamente hacia aguas abajo	6. Obras de protección en todo el ancho tanto del río, tanto hacia aguas arriba como hacia aguas abajo	PPR12	0
Puntuación del Periodo de Retorno [año]	0	1	2	2	5	15		
Materiales predominantes del cauce en el río	1. Limo/Arcilla		2. Arena	3. Grava	4. Roca/ Cantos Rodados		PPR13	0
Puntuación del Periodo de Retorno [año]	5		10	5	10			
Existencia de troncos caídos, basura y/u otro tipo de escombros	1. Reconocido		2. Escasamente reconocido			3. No reconocido	PPR14	0
Puntuación del Periodo de Retorno [año]	0		5			10		
Condiciones de entorno del río (vista en perfíl longitudinal)								
Degradación del cauce del río	1. En todo el ancho		2. Parcial		3. No aplicable		PPR15	0
Puntuación del Periodo de Retorno [año]	0		1		10			
Agradación del cauce del río	1. En todo el ancho		2. Parcial		3. No aplicable		PPR16	0
Puntuación del Periodo de Retorno [año]	0		1		10			
Diferencia de Gradiente de Pendiente del cauce del río (DGP) en el puente, dentro de una longitud de 100 m, hacia aguas arriba y aguas abajo.	1. Si aguas abajo es más pronunciada que aguas arriba			2. Si aguas arriba es más pronunciada que aguas abajo			PPR17	0
	1. DGP < 10%		2. 10% ≤ DGP	3. DGP < 10%	4. 10% ≤ DGP			
Puntuación del Periodo de Retorno [año]	20		10	10	15			
Altura de caídas con o sin obras de protección	1. H < 1 m		2. 1 m< H ≤ 2m	3. 2m< H	4. 5m < H		PPR18	0
Puntuación del Periodo de Retorno [año]	10		5	2	0			
Daños en obras de protección en caídas	1. Mas del 50% del ancho o no existen		2. Menos del 50% del ancho		3. No aplicable		PPR19	0
Puntuación del Periodo de Retorno [año]	1		3		5			
Daños Estructurales en el Puente								
Daños (tales como grietas abiertas, socavación) en subestructura (para puentes colapsados, antes de situación de desastre)	1. Daños Severos	2. Daños Moderados	3. Sin daños o con daños menores		4. Desconocido		PPR20	0
Puntuación del Periodo de Retorno [año]	0	5	10		0			
Daños (tales como grietas abiertas) en superestructura (para puentes colapsados, antes de situación de desastre)	1. Daños Severos	2. Daños Moderados	3. Sin daños o con daños menores		4. Desconocido		PPR21	0
Puntuación del Periodo de Retorno [año]	0	5	10		0			
Probabilidad Anual de un Evento de Desastre Puente = Σ (PPR1: PPR21), sumando las puntuaciones anuales de probabilidad								**0**

Fig. 3-5 Vulnerability Assessment sheet on Bridges (Continuation)

3.7. REFERENCES

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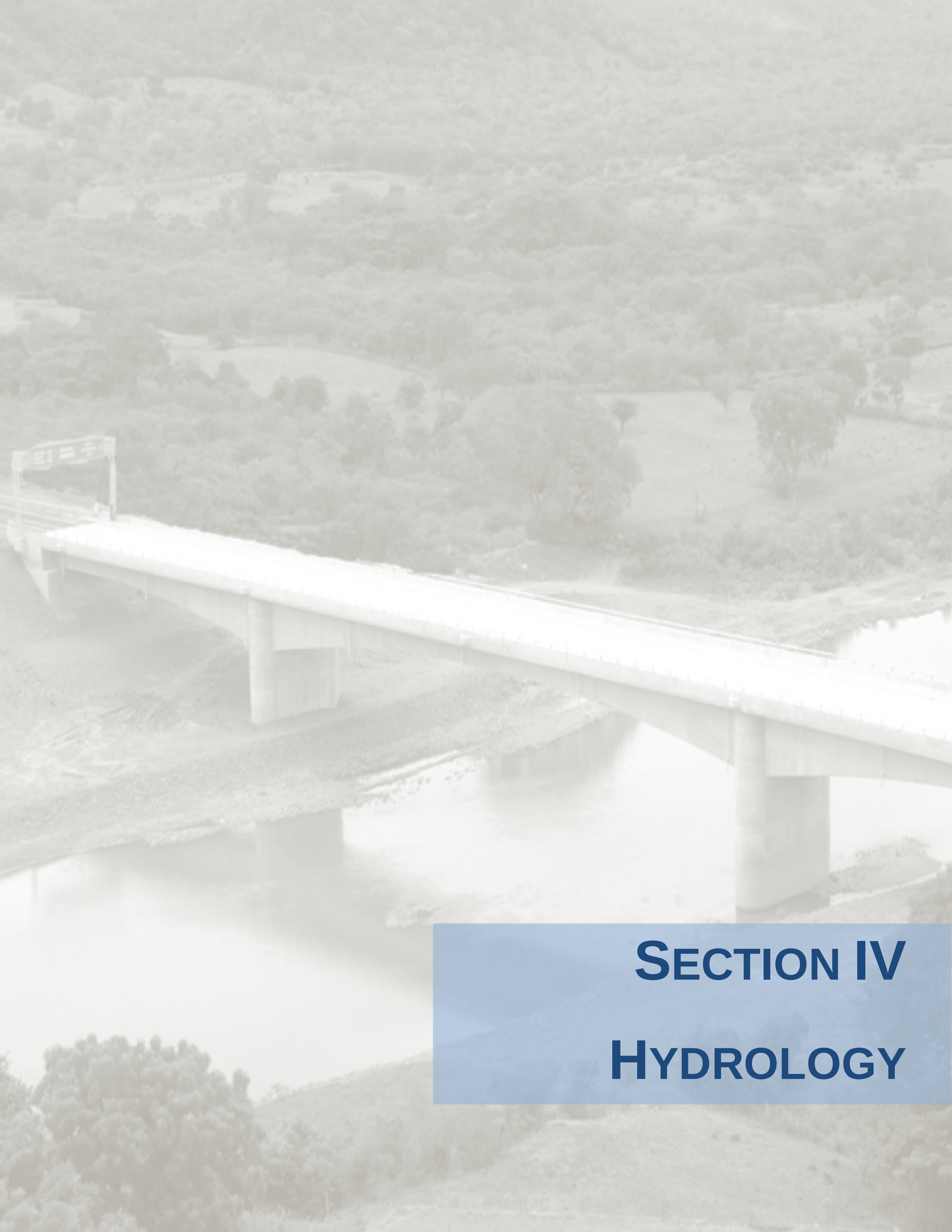


Section IV

Hydrology

HYDROLOGY

Hydrology is the science that studies the characteristics and distribution of water in the atmosphere, on the surface of the earth and in the soil, the hydrologic cycle can be considered one of the main basic concepts; in this, rainfall is the cause of the flow in the rivers.

That is why one of the basic components of the water cycle studied in hydrology is the rainfall that occurs over a specific geographical area. In order to study it, it is necessary to define the basic control unit: hydrographic basin.

The basin is an area of land area where the raindrops falling on it tend to be drained by the system of flows to the same starting point. (Aparicio, 2005). The location of basin partially determines the climatic conditions that give rise to the meteorological phenomena that constitute the core of hydrology. However, the physical characteristics of the basin not only control the hydrologic response to meteorological phenomena but also some features, like orography and aspect, they may also be factors that determine the climate of the basin (World Meteorological Organization, 1994).

The hydrologic analysis of the basin constitutes an important preliminary step in the drainage structures design on roads, since they are designed to drain certain flows that avoid possible effects on infrastructure or the environment.

In the process of estimation of these flows, hydrology specialists are primarily interested in three properties of rain:

- The rate at which it falls on the ground, known as Intensity.
- The length of time for a given intensity, known as duration.
- The probable number of years elapsed before a combination of intensity and duration given will repeat, known as frequency.

This section is a collection of basic hydrologic aspects to be considered for estimating the maximum flow rate used in the design of a drainage work on roads. Also, the tools of probability and statistics applied in studies of hydrologic data. The methodologies described are drawn from different literature sources; to delve into each of the topics exposed, it can be drawn upon to the references listed in this section or another concerning hydrologic analysis.

4.1 WORKING SCALE

To carry out the hydrologic analysis it is necessary to start with the determination of physiographic characteristics of the basin. This process requires diverse, mostly topographical information, which is available in different scales in the region.

According to the environmental conditions and the type of structure to be designed, it is advisable to establish a minimum level of work for the determination of physiographic features. Authors like Fattorelli y Fernandez suggest a range of scales depending on the basin area of study, which can be seen in Table 4-1. Worth remembering that in the case of minor drainage design, the need to perform a topographic survey of the drainage area should be evaluated.

Table 4-1 Recommended work scales for different surfaces of basins (Fatorelli & Fernández, 2011).

BASIN AREA	RECOMMENDED WORKING SCALE
Under 100 km²	1:25000 to 1:50000
From 100 km² to 1000 km²	1: 50000 to 1: 100000
From 1000 km² to 10000 km²	1: 100000 to 1: 250000
Larger than 10000 km²	1 : 250000 to 1: 500000
Larger areas	1: 500000 to 1: 1000000

4.2 PHYSICAL CHARACTERISTICS OF THE BASINS, DRAINAGE AND MAIN STREAM

Although there may be many characteristics of the basin, the drainage pattern and the main stream to be described, this document will refer primarily to those that affect the surface runoff and, therefore, the estimation of the maximum flow for the design of drainage works on roads.

As regards to the basin, it is important to bear in mind its size, shape, slope, soil permeability, capacity of underground water formations, presence of lakes and swamps and land use. Furthermore, the characteristics of the main riverbed are referred to these hydraulic properties, which determine the flow movement and sewage capacity of the channel.

The following describes the main physical characteristics to be considered in defining a basin and influence the hydrologic response of this:

4.2.1 Basin area or drainage area.

Understood, as the area bounded by landforms for which the surface water volume drains, is a property that contributes, together with other properties, to the shape of the basin response to rainfall. This property is part of the basic parameters in a hydrologic study.

The drainage area is mainly produced from drawings, maps, aerial photographs and topographic maps. Direct field measurements become preferable especially for smaller basins, as they generally do not have enough information or topographic maps on a large scale. In addition, a field study may determine such alterations that over the years have caused anthropogenic activities in the relief of the basis, which are not recorded in the official topographical information.

Other tools such as Digital Terrain Models (MDT), also known as Digital Elevation Model (MED), along with the Geographic Information Systems (SIG) are useful and accurate; however, it is not possible to be found throughout the region. Although, it is possible to consult free geo services, such as the geoportal of the Consortium for Geo Spatial Information (CGIAR-CSI) which provides digital elevation models all over the world. This information can be found at the following website: http://srtm.csi.cgiar.org/

4.2.2 Basin Perimeter

It is defined as the length of the basin delimited by the drainage divide; it is obtained in the same manner that the area. In cases where the drainage divide is too sinuous, drawing a line of best fit to represent the partition can be chosen.

4.2.3 Maximun and minimum height, grade and hypsometric curve

The maximum elevation and average height are indicators that determine the extent to which air masses must rise to pass over the basin.

The maximum height is a value that is read directly from the planimetry of the basin, placing the tallest elevation in the basin studied. The minimum height is determined in correspondence with the level of the main stream in the control section. While the height difference is the subtraction the maximum height and minimum height.

The hypsometric curve reflects with greater accuracy the overall performance of the altitude of the basin. It represents the percentage or fraction of the area of the basin is above a certain elevation or lifting intervals. For its construction, is placed on the abscissa axis, the percentage of total area located above the elevation provided on the ordinate axis. These percentages of graphed accumulated area may be obtained by planimetric calculations of successive surfaces between the contours.

The hypsometric curve can be useful in identifying physical and behavioral characteristics of the basin. For example, steep gradients in the upper parts that then decrease in the lower elevations may be an indication of areas susceptible to flooding.

Also, from this curve can be drawn a significant relationship:

$$R_h = \frac{S_s}{S_1} \tag{4-1}$$

Where:
S_s, is the area on the hypsometric curve.
S_1, is the area under the hypsometric curve.

According to Strahler, this ratio is an indicator of the dynamic balance of the basin. Thus, when the value of Rh is approximate to one there is a basin with morphological balance. For different values, the interpretation that might be performed shown in Fig. 4-1.

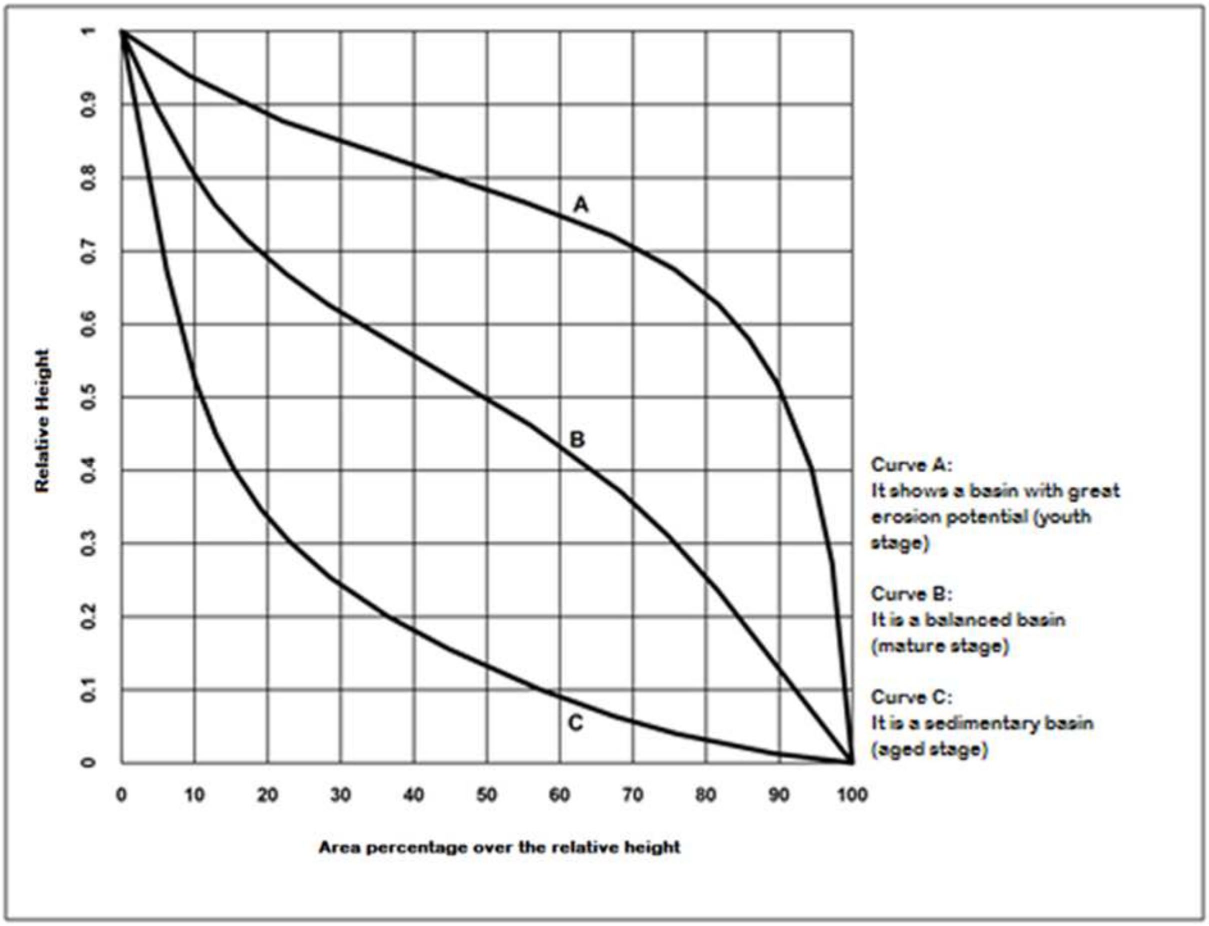


Fig. 4-1 *Characteristic hypsometric curves of a Basin, according to Strahler.*

4.2.4 Basin Slope

It is an important factor in estimating the time of surface runoff and time of concentration of the watershed. Its importance is reflected in the rainfall-runoff relationship, which, increasing the gradient results in decreased concentration time (which is defined as the minimum time required for all points of a basin to contribute runoff water simultaneously to the starting point) and higher discharge peaks. In addition, infiltration volumes are reduced, increasing the volume of water surface. Inversely it happens with decreasing.

If a GIS and and a Digital Elevation Model of the Terrain are available, calculating the slope is almost immediate. In case of not having these tools, one of the criteria that can be used is that of Horton, in which the gradient can be calculated using contour maps, through a graphic-analytical method vertically and horizontally, as follows:

First, placing on the mapping and altimetry of the basin a regular grid (the accuracy of the results depends on the dimensions of the grid), Fig. 4-2, the points of intersection of the vertical lines are counted with any contour line (red dots); then, the length of the vertical stretches if the grid (green lines) are measured within the boundaries of the basin and the following formula is used:

$$P_{vert} = \frac{n \, . \, e}{\sum L_{vert}} \tag{4-2}$$

P_{vert}, is the measure of gradient vertically.
n, is the number of intersections.
e, the distance between contour lines in meters.
ΣL_{vert}, is the sum of the lengths of the vertical grid, in meters.

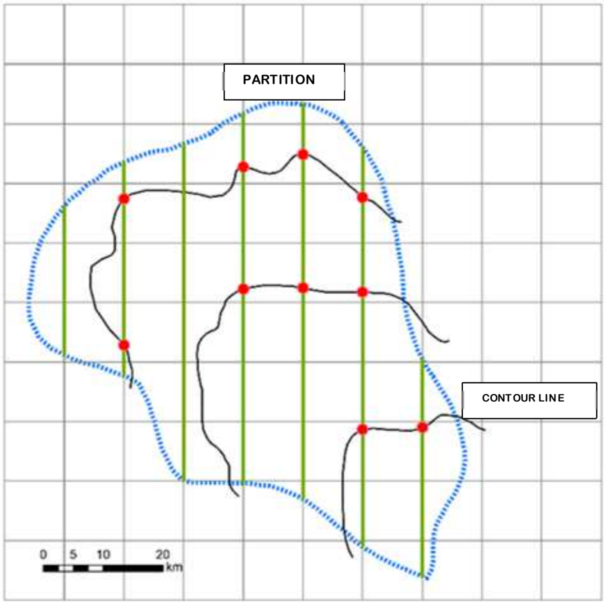


Fig. 4-2 Methodology for determining the average stretch of the basin in the vertical direction (Román, 2013).

Then the same procedure is performed in the perpendicular direction to obtain the stretch in the horizontal direction (P_{hori}).

The average stretch (P_m) from the basin will be the arithmetic average of the results obtained in the vertical direction and the horizontal direction.

$$P_m = \frac{P_{vert} + P_{hori}}{2} \tag{4-3}$$

Another approximate method for the calculation of the gradient, and suggested by WMO, is by estimating an average gradient obtained by the formula:

$$P_m = z.\frac{\Sigma \, \mathrm{L}}{A} \tag{4-4}$$

Where:
z, is the interval of the contour lines.
$\sum$L, It is the total length of all contour lines within the basin.
A, is the area of the basin.

The main difficulty when using this formula is presented in the estimation of the total length of the contour lines ($\sum L$); since direct measurement of these on the topographic map can drag major inaccuracies, especially in curves that are very broken, which are necessary to be smoothened.

4.2.5 Compactness rate or Gravelius compactness coefficient

Rate representing the plan shape of the basin studied. Relates the perimeter with the perimeter of a circle of equivalent area to the area of the basin. Since the circle is the geometric figure with the smallest perimeter, the compactness rate is always greater than 1, and as this is closer to the unit, it will indicate a more rounded basin.

La expression to obtain the Compactness Rate is as follows:

$$\mathrm{Ic} = \frac{P}{2\sqrt{\pi . A}} = \frac{0.28\,P}{\sqrt{A}} \qquad (4\text{-}5)$$

Where:
P, is the perimeter of the basin
A, is the basin area.

For a basin with increasing compactness coefficient, the time of concentration will be greater. Hence, it is expected that the magnitude of runoff generated by rainfall in it is lower than in one that has a lower coefficient of the compactness. The shape of the basin is an indicator of how storms and the runoff pattern of the basin are spatially distributed. An elongated shape means that water runs through various tributaries to reach a main stream, which results in a slower response.

According to the compactness rate value, classifications can be made on the elongation of the basins. An example of this circular is to define them as if they have a value between 1.0 and 1.25; ovate between 1.25 and 1.50; oblong, between 1.50 and 1.75; rectangular oblong, between 1.75 and 2.0; elongated rectangular, more than 2. It should be noted that this classification is not unique; other references may use different values to catalog basins.

4.2.6 Shape Factor

It is a feature that mostly affects the rate of velocity at which the flow reaches the main stream and then the site of interest.

Basins with more long and narrow shapes, compared on equal terms with other wider, have lower discharge peaks. Likewise, basins, which its center is farther from its discharge point, have lower peaks, i.e., as this distance is shortened, discharge peaks increase. Therefore, the importance of defining this factor shows that the highest peak moves a certain volume in less time, which makes it necessary to consider this phenomenon in the design of the drainage structure. Therefore, this situation should warn the designer about future conditions of the works to be built. One effect that can occur by this phenomenon is the increased flow caused by backwater, also depending on the initial conditions of the work.

As for the rainfall management, when it moves transversely to the axis of the elongated basin, runoff rates are lower than when rainfall progresses lengthwise to the axis.

The shape factor (Kf) is the ratio of the average width of the basin (B) and the length of the main river course (Lc). While the average width is the ratio of the area of the basin (A) and the length of the main runway. So:

$$K_f = \frac{A}{{L_c}^2} \tag{4-6}$$

Where:
A, is the area of the basin.
L_c, is the length of the main riverbed of the basin.
A basin with low shape factor is less subject to a flood than one with the same area but greater shape factor.

4.2.7 Land Use

Land use study is needed to define the conditions presented in the basin and estimate future conditions. The conditions of land use affect the hydrology of the area, which represents changes in the maximum flows, volumes of runoff, water quality, etc.; this shows that changes in land use affect the response mode of the basin to rainfall.

The most frequent changes in land use are due to urbanization, which means a reduction of waterproofing areas infiltration; before this, decreases in time concentration were observed and therefore, higher discharge streams downstream.

Factors such as urbanization and agricultural practices have the greatest impact on small basins. However, for an overview the impact caused, it is necessary to calculate the maximum flow post urbanization and compare it to stream in the natural state of the basin.

Currently, land use classification can be made through remote sensing using satellites images. For example, the Landsat sensors widely used for this type of study are available free on the site of the United States Geological Service: http://earthexplorer.usgs.gov/.

4.2.8 Land and Geology

The type of soil has a direct effect on the ability of infiltration and surface runoff; this varies according to the condition, in which the soil is when precipitation occurs, but also the magnitude and intensity of rainfall.

Likewise, soil formations underlying the surface layer and the presence of groundwater reservoirs; affect the response of the basin to rainfall.

4.2.9 Sewage area – volume

The storage in basin is based on: Intercept storage, which is the portion of rainfall that is intercepted by vegetation and never touches the ground to become surface runoff; storage in small depressions in the land surface; storage in the superficial transit or piping and storage in lakes, ponds and swamps[1].

Storage by interception or depressions on the surface may be negligible for engineering work analysis such as roads, but not in urban drainage works, in which the calculation becomes finer.

The effect of water stored in the maximum flow rate may be negligible for sewage basins under 1% of the basin area. (AASHTO, 2006).

4.2.10 Basin Orientation

The orientation of the basin is a secondary factor study, but certainly not negligible; its influence lies in the losses caused by evaporation and transpiration by the heat received from the sun.

Likewise, the orientation of the basin with respect to the direction in which the storm moves has affectations in the peak discharges of the basin, as mentioned in section 4.2.6. A storm traveling length wisely in the direction of flow of the basin will produce greater peak discharges and less runoff periods. Although rainfall travels from the discharge point of the basin, rising upstream, the effect will be less.

4.2.11 Riverbed configuration and floodplains geometry

These are characteristics that directly affect the surface and subsurface runoff, both in volume and in the transport velocity.

Factors to consider in the study of the configuration of channels and floodplains are the channel sinuosity, the channel cross section, its system of effluents, the channel storage, the density of the vegetation of the channels and floodplains. All of these factors in an integrated manner affect the discharge rate of the basin.

4.2.12 Drainage density

It is an important indicator of the shape of the land and the degree of erosion that may have the basin in terms of geological, vegetation and type of soil. The drainage density (Dd) can be expressed as the ratio of the total sum of the lengths of all riverbeds of the basin per unit area.

$$D_d = \frac{\sum Lc_i}{A} \qquad (4\text{-}7)$$

Where:
$\sum Lc_{i,}$ is the total sum of the lengths of all riverbeds of the basin in km.
A, is the total basin area in km^2.

[1] In Honduras, surface water bodies are generally known as aquifers.

In some cases, the drainage density does not provide a true measure of the efficiency of drainage. However, in general, it reflects the magnitude of the potential flooding.

In general terms, the higher the density values of drain, the greater the peak and the total volume of runoff. Generally, the values ranging from 0.5 km / km^2 for poor drainage basins, up to 3.5 km / km^2 for well-drained basins. (Fatorelli & Fernández, 2011)

4.3 HYDRO METEOROLOGICAL INFORMATION ANALYSIS

In the development of a hydrological study, rainfall is the cause of water on the earth surface, so measures must be made to determine the physical, chemical and mechanical properties of the water from the basin or basins to be treated for use and control.

Measurements are made through a series of data that will need to be processed to obtain useful parameters for designing engineering works. Said process is conducted under statistics laws and probability methods that apply to data, which it has. Statistics is dedicated to the display, description and summary of data and model generation from data obtained from a sample, while the probability studies the possibility of occurrence of values equal to those of the sample.

In hydrology, discrete or continuous data can be obtained, depending on the phenomenon being studied; time series, which are those statistical data observed and recorded at regular time intervals, are set by the occurrence of a natural event.

Data types, which are usually obtained in hydrology, are grouped into:

- Historical data of natural events are recorded in a chronologic, direct or continuous manner. They are time series resulting from observations lost if not recorded at the time of its occurrence. To this type it belongs the vast majority of hydrologic and hydro meteorological data.
- Field data of eventual form or for a specific purpose, such as depth and quality of underground water, infiltration or sedimentation in rivers.
- Data measured in the laboratory, usually referring to the physical-chemical water quality.
- Simultaneous recording of an event (rainfall-runoff) in two different geographic locations, for a certain period of time (usually 4 or 5 years) used to transfer information to correlate data for various purposes, as flow analyses are.

Ideally, it is expected from the data obtained, to be independent, homogeneous and representative of the basin study. However, the process of obtaining is not free of errors at the time of implementation.

These errors may be random errors, which are located in the data itself and usually are distributed around the actual value of the measured quantity; they can be estimated by the standard deviation, which determines the degree of variability of the magnitude.

The other errors are systematic, and are related to the accuracy of measurements. They may be caused by defects in the measuring instrument or the measuring process, among other things; they generate inconsistencies describing a pattern, which makes them identifiable and correctable using different methodologies.

It can also be accompanied by the possibility of having non-homogeneous data due to specific phenomena, which change the normal bias of a data series. The cause of the inhomogeneity may be an abnormality in the rainfall season for some period, such as a change in location of the season, changing the conditions of the recording device or changes in the method of construction. In these situations, it is imperative to recognize the cause of the inhomogeneity to determine what action to take.

4.4 RAINFALL DATA ANALYSIS[2]

4.4.1 Average rainfall

One of the first steps in analyzing hydro meteorological data is to determine the height of rainfall in a given site. Since this point differs in its spatial distribution and measuring instruments record data at specific points in space, it is necessary to draw on methods to facilitate the determination of the average rainfall for a given or part of basin from a storm.

Methodologies widely used for the determination of the average rainfall are:

a) Arithmetic Method

Consists of getting the first moment around the origin or the arithmetic average of the heights of rainfall recorded at each site where a season is located.

The formula for obtaining the average rainfall is:

$$h_p = \frac{1}{n}\sum_{i=1}^{n} h_{pi} \qquad \textit{(4-8)}$$

Where:
h_p, is the average rainfall.
h_{pi}, is the rainfall recorded in rain gauge *i*.
n, is the total number of rain gauges within the basin.

This method is the simplest to implement, but does not take into account the distribution of rain gauges and the spatial distribution of rainfall; also it assigns equal weight to all the rainfall values registered. Therefore, for widely scattered extreme values the value of the average rainfall is not very representative of the basin.

It is recommended to use this method in areas with gentle topography and uniform atmospheric conditions, or to obtain a first reference to the height of the average rainfall in the basin.

[2] Fundamentals on Surface Hydrology, Aparicio, 1989

The accuracy of this method depends on the amount of available seasons of how are located and the distribution of rainfall studied.

b) Thiessen polygons

This method is widely used for the determination of the average rainfall in a basin. Take into account the distribution of rain gauges within it; therefore, it is necessary to know the location of the same to identify the area of influence of each of them within the set of stations. This method does not take into account the topography of the basin; so its use is recommended for areas with no topography. From the practical standpoint, represents a saving in time when it is necessary to analyze a large number of storms because polygons do not change shape. Unless added or removed any of them, and this method is more accurate than the simple method because it considers the area of influence of each station.
The methodology consists on applying the following steps:

- After obtaining a map with the location and distribution of the rainfall stations inside and outside the basin, attached by straight lines the nearest stations from each other; trying to form triangles, if possible with angles less than 90° with vertices found in rainfall stations.
- Draw bisectors to the lines connecting the rainfall stations. These bisectors intersect at a point inside the triangles formed in the previous step.
- Extend bisectors to the limits of the basin.
- Each rainfall station will be surrounded by a polygon, called “Thiessen Polygon”. The surface of the polygon represents the area of influence of each corresponding station.
- The average rainfall is calculated by the weighted average of the observed rainfall in every season. The weighting factor will be the area of influence for each station as shown in the following expression:

$$h_p = \frac{1}{A_T}\sum_{i=1}^{n} A_i\, h_{pi} \qquad (4\text{-}9)$$

Where:
A_i, is the partial area of the Thiessen polygon corresponding to each gauge *i*.
A_T, is the total area of the basin.
h_{pi}, precipitation recorded at rain gauge *i*.

c) Isohyets method

An isohyet is a curvaceous line representing points of equal rainfall in a geographical area, analogous to the contour lines determined in topography.

In mountainous regions, the method of isohyets is recommended for calculating the average rainfall. If rainfall is orographic, it will have to follow a similar configuration to the contour lines.

The method can be very accurate when considering these topographic effects. On the contrary, accuracy is not greater than the Thiessen Polygons method if a linear variation of the value of rainfall is assumed between stations.

In addition, the greater the number of stations used for analysis, the better the accuracy obtained.

The expression to calculate the average rainfall is as follows:

$$h_p = \frac{1}{A_T}\sum_{i=1}^{n} A_i \, h_{pi} \qquad (4\text{-}10)$$

Note that this expression is equal to that used with the Thiessen polygons method, only in this case, the weighting factor Ai is the area between two consecutive isohyets and the partition of the basin, and h_{pi} is the height of average rainfall between two isohyets.

After determining the average rainfall in the basin or in the area affected by the project, it will be necessary to find a way to determine the accuracy of the estimate made. Some authors estimate that the standard error in the calculation of the average rainfall rate can be evaluated using the following formula developed by the World Meteorological Organization, WMO, and referenced by Collado and Domínguez:

$$E = aA^bN^c \qquad (4\text{-}11)$$

Where:
b and *c*, are constants that can be approximated as 0.2 and -0.5, respectively, and depends on the characteristics of the basin and storm.
A, is the area of the basin.
N, is the number of stations for analysis.

4.4.2 Filling of missing data in rainfall sets of data

For different reasons in Central America, rainfall records are not always complete. For these cases, there are methodologies that aim to estimate the missing data if they have simultaneous records of other stations near the station in question.

In the graphs of Fig. 4-3 is shown an alternative to perform the estimation of missing data. The first option is to correlate the average rainfall at a nearby station to the station in question (Fig. 4-3a). The second is to make the correlation with the average of the measurements in several surrounding stations. (Fig.4-3b). The correlations are performed through the application of regression models they are not necessarily linear.

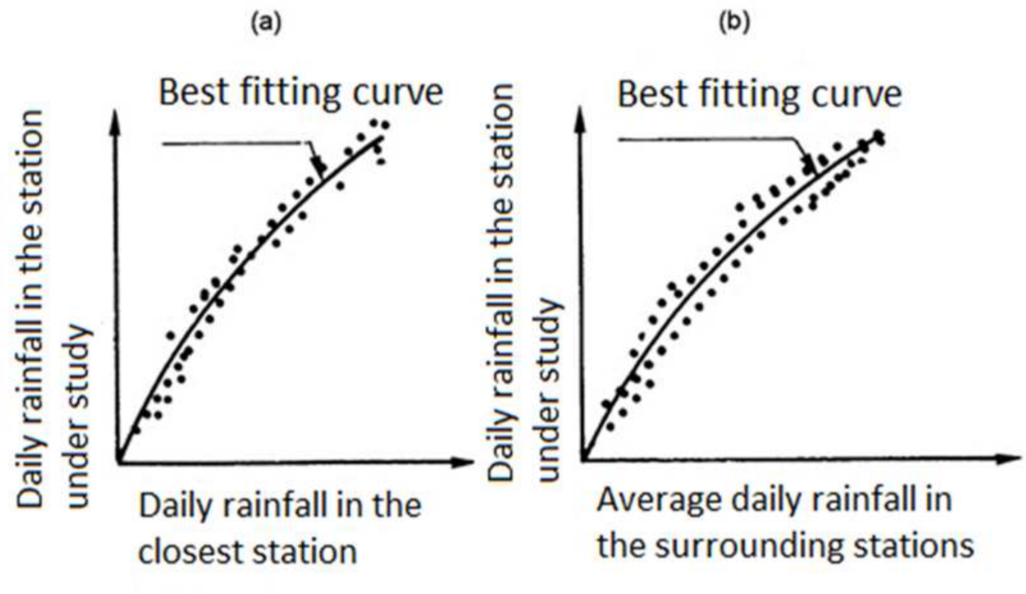


Fig. 4-3 *Correlation of rainfall data from a station with missing data with a station nearby (a) and several surrounding stations (b) (Aparicio, 1989).*

When obtained the return equations with their respective graphs, in which the x-axis data of the reference stations and the y-axis data of the station in question is located, it is verified that the correlation of data is acceptable. A correlation coefficient of 0.8 or higher is considered acceptable, although not a final judgment because it also depends on the number of pairs of data with which work has been done and an estimation error has been established.

When obtained a good correlation, just enough to release data from nearby stations in the days in question to estimate the missing data in the analysis station.

If the correlation is not considered permissible or acceptable, based on the average annual rainfall method it can be applied to estimate the missing data. Two cases to be considered:

- If the annual rainfall in the reference stations differs by less than 10% with the station in question, the missing data are estimated by obtaining the average of the data reference stations.
- If the difference between the data from the reference stations and station study differ by more than 10%, the following formula should be used:

$$h_{px} = \frac{1}{n}\left[\frac{p_x}{p_1}\,h_{p1} + \frac{p_x}{p_{12}}\,h_{p2} + \cdots + \frac{p_x}{p_n}\,h_{pn}\right] \qquad (4\text{-}12)$$

Where:
h_{px}, is the missing rainfall in the station in question.
h_{pi}, is the rainfall registered the day in question in the reference station i.
p_i, is the annual rainfall in the reference station i.
p_x, annual rainfall in the station in question.

n, number of reference stations.

The number of reference stations used for the application of this formula must be at least 3.

4.4.3 Records adjustments due to the lack of homogeneity in data

When the bias of the records suffers deterioration due to external factors or changes in measurement conditions, adjustments are necessary to detect and correct any alterations in the data.

The development of the double mass curve is a technique that is based on changes in the average accumulated rainfall of various seasons are not sensitive to changes in one, because of the errors that may arise, are compensated.

To develop the double mass curve, in the x-axis the average annual accumulated rainfall of the reference stations is placed, therefore, it is appropriate that the number of stations to use is not less than ten for better results; then in the y-axis, the cumulative annual rainfall analysis station is placed.

The expected result is a straight line, as long as no major changes have occurred; otherwise, the line will change of gradient in the year in which the measurement conditions changed.
Then, to ensure that data is consistent at the station in question, previous records to the year when the changes were detected must be multiplied by an adjustment factor.

In Fig. 4-4 illustrates the adjustment made to a series of annual data. You may notice that as of 1976 there is a change in the gradient of the line (solid line). The correction factor before the 1976 data will have to be made based on the following ratio:

$$Fc = \frac{0.83}{0.63} = 1.32 \tag{4-13}$$

Therefore, the values have to be multiplied by 1.32.

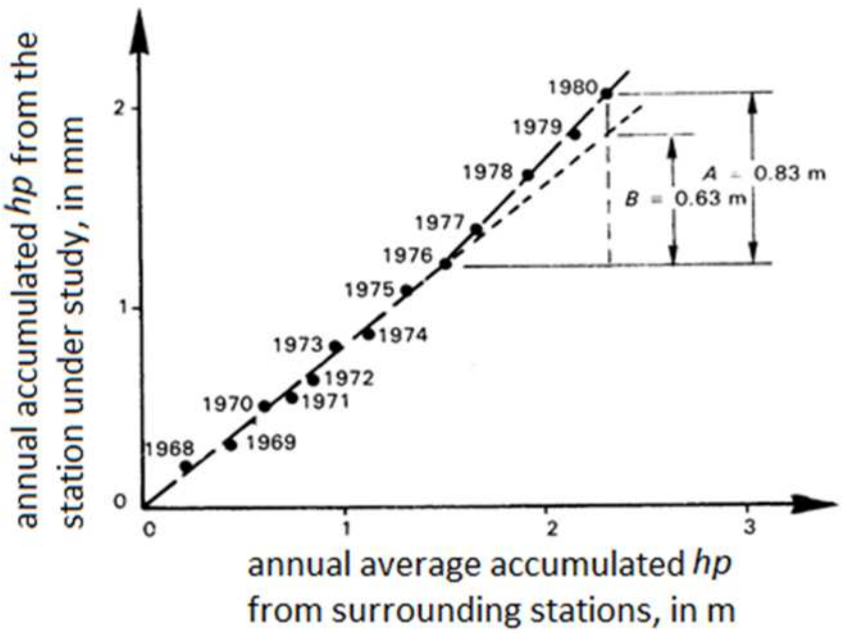


Fig. 4-4 Determination of the correction factor for a series of annual data (Aparicio, 1989)

4.4.4 Development in Intensity-Duration-Frequency (IDF) curves

Statistical analysis of the data obtained from a hydrologic basin is useful for determining the risk involved in proposing the design parameters of a work. Of these, the precipitation data are useful for feeding a rainfall-runoff model, which results in obtaining the design flood in the future.

The IDF curves are a graphical tool showing the relationship between the three parameters of the rain of interest in the design of drainage works: the rate at which it falls on the ground, known as intensity, the time for a given intensity, known as duration, and the possible number of years to elapse before a combination of intensity and duration given will repeat, often known as frequency.

The methodologies for determining the relationship between the intensity, duration, and frequency of rainfall are two:

The first relates the intensity and frequency for each term separately by a probability distribution function used in hydrology. It is known by the name of intensity-return period.
The second, which is taking place in this document, relates the variables of intensity, duration and frequency simultaneously in a family of curves through the following expression:

$$i = \frac{kT^m}{(d+c)^n} \qquad (4\text{-}14)$$

Where:
k, m, n and c, are constants calculated by a multiple linear correlation.

To use the formula, it is necessary to have records maximum rainfall heights in mm, for different durations of rain. The maximum rainfall heights are normally associated with only one or two of the greatest storms of the year.

The most convenient is to have records of more than 25 years to get a good reliability in the analysis. In case of having fewer years of records, the specialist will decide the representativeness of the sample and / or the method to use.

The steps for determining the IDF curves are as follows:

- Transform the heights of rainfall intensities, dividing said height between the respective duration and express the intensity in mm / h.
- When data is transformed to intensities, it is needed to assign them a return period (see section 4.4.5) and sort the data for each duration from high to low.
- Applying to both sides of the equation of intensity, shown at the beginning of this paragraph, the natural logarithm, we will have the following equation:

$$\log i = \log k + m\, logT - n \log(d + c) \qquad (4\text{-}15)$$

Doing analogy with the equation of the form:

$$y = a_0 + a_1 x_1 + a_2 x_2 \qquad (4\text{-}16)$$

Representing a family of straight lines of gradient a2 the intercept a_0 and spacing a_1. We will have: y = log i; a_0 = log k; a_1 = m; x_1 = log T; a_2 = -n; x_2 =log (d + c)

If data from i, d y T at graphing them on logarithmic paper, they are grouped around straight lines, the value of c can be taken as zero.

- Then an adjustment of multiple linear correlation must be applied to the series of three types of data and a system of equations is obtained as the following:

$$\sum y = N\, a_0 + a_1 \sum x_1 + a_2 \sum x_2 \qquad (4\text{-}17)$$

$$\sum (x_1 y) = a_0 \sum x_1 + a_1 \sum (x_1^2) + a_2 \sum (x_1\, x_2) \qquad (4\text{-}18)$$

$$\sum (x_2\, y) = a_0 \sum x_2 + a_1 \sum (x_1 x_2) + a_2 \sum (x_2)^2 \qquad (4\text{-}19)$$

Where N is the number of data; a_0, a_1 and a_2 are the unknowns and x1, x2 and y are the logarithms of return period, the logarithm of the duration and the logarithm of the intensity, respectively.

Resolved the equations and calculated the coefficients a_0, a_1 and a_2, it is possible to determine the parameters k, m and n and draw the curves.

4.4.5 Return Period

Since a series of hydrologic records, it is important to investigate the flow producing basins in order to design drainage works on roads. For this, it is necessary to analyze the frequency of such records in order to determine the return period of an event of a given magnitude.

In the case of rainfall data, which are the result of a random event, analysis and prediction it is to be performed through the application of probabilistic concepts. So it is necessary to assign an experimental frequency to each of the elements of a series based on the order according to their magnitude (in ascending order for low value frequencies, and descending order for high value frequencies), assigning positions 1, 2, 3 to n.

The term mostly used for assigning the frequency, is the Weibull formula (1939):

$$\mathrm{P(x)} = \frac{\mathrm{m}}{\mathrm{n}+1} \qquad \text{(4-20)}$$

Where:
m, is the position that is assigned to an event in the ascending order of magnitude.
n, is the sample size (n values of rain or n flow values).

The inverse function of P(x) is known as the return period or recurrence interval (T_r) of an event. Which it is defined as the average time between events that meets or exceed a given magnitude or, in other words, the time interval within which an event of a given magnitude can be matched or exceeded. The return period does not determine the exact time of occurrence of an event.

It is expressed as follows:

$$T_r = \frac{1}{P(x)} \qquad \text{(4-21)}$$

If the sample of the variable refers to a year, a month or a specific time, T_r will be referred to this period. (Yevjevich, 1972)

Another way to calculate P(x) is given in the California method (1923):

$$\mathrm{P(x)} = \frac{\mathrm{m}}{\mathrm{n}} \qquad \text{(4-22)}$$

$$\mathrm{P\ (x)} = \frac{\mathrm{m}-1}{\mathrm{n}} \qquad \text{(4-23)}$$

When there are partial series and high values, Hazen formula (1914) can be applied, as it gives intermediate position values between the values given by the method of California.

$$\mathrm{P(x)} = \frac{\mathrm{m}-0.5}{\mathrm{N}} = \frac{2\mathrm{m}-1}{2\mathrm{N}} \qquad \text{(4-24)}$$

Other formulas that can be used are Beard (1943), Blom (1958), Cunnane (1978) and Adamowski (1981).

4.4.6 Risk Analysis[3]

There are certain situations in which the designer wants to know the probability of occurrence of a given event for a specific time, for example increasing the probability associated with a certain T_r, during the construction of the road.

The probability P that a flood with an average occurrence probability P is exceeded exactly x times over a year period n is given by the following expression:

$$P = \binom{n}{x}(1-p)^{n-x}q^x \tag{4-25}$$

Where:

q, is the complementary probability:

$$q = 1 - p \tag{4-26}$$

y combinatorial number is given by:

$$\binom{n}{x} = \frac{n!}{x!\,(n-x)!} \tag{4-27}$$

This expression of probability is based on the binomial distribution or "repeated tests" from Bernoulli. The classic example of this type of test is the toss of the coin: the given expression allows to determine what is the probability of getting x heads in n throws. The average probability p in this case is to obtain a face on a random toss (equivalent to the average probability of a flood or return period); this value is 0.50 as well as to the tails (in other words, over a sufficiently large number of throws, the same number of heads and tails would be obtained).

Having that $p = 1/T_r$ as defined by the return period for the particular case when $x = 0$, we have the probability of occurrence of the phenomenon:

$$P_0 = \left(1 - \frac{1}{T_r}\right)^n \tag{4-28}$$

into n years, being n the lifetime of the work.

The probability of occurrence of the phenomenon at least once will be complementary to the previous:

$$\mathrm{R} = 1 - \left(1 - \frac{1}{T_r}\right)^n \tag{4-29}$$

probability that is usually defined as the risk of occurrence of design flood in the lifetime of the work.

[3] Roads Manual: Drainages and Bridges, Public Works, Transport and Housing Secretariat, SOPTRAVI, Honduras 1996

Table 4-2, was made from the above expression it can be used to determine the probability of occurrence of an increasing recurrence given at a specific period.

Table 4-2 Probability of an event of a period of recurrence is equaled or exceeded, for different durations (Secretary of State in the Ministry of Public Works, Transport and Housing, 1996)

n	1	5	10	25	50	100	200	500
T_r	R							
1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2	0.50	0.97	0.99	*	*	*	*	*
5	0.20	0.67	0.89	0.996	*	*	*	*
10	0.10	0.41	0.65	0.93	0.995	*	*	*
50	0.02	0.10	0.18	0.40	0.64	0.87	0.98	*
100	0.01	0.05	0.10	0.22	0.40	0.63	0.87	0.993
200	0.005	0.02	0.05	0.12	0.22	0.39	0.63	0.92

* In these cases R can never be 1.00, but for all purposes it can be taken as the unit.

It can be seen in the above table, the values of risk R in n years of a frequency event T_r; For example, if a culvert has a useful life of 10 years and is projected to evacuate a 50 year flood, there is a 18% risk that their useful life is subjected to a flood equal to or greater than the designed one.

4.4.7 Functions of probability distributions [4]

In most cases the return period for a structure to be designed exceeds the period of hydrologic records collected and the problem lies in how to extend the trend (maximum values - return periods) to a desired period. Therefore, it is necessary to extrapolate from the maximum values of rainfall to estimate the maximum flow of the basin of the required return period.

To find a solution to the problem, there are theoretical probability distributions to match the measured data and can be used. The selection of the function depends on physical considerations of the basin studied, previous treatment experiences of the same variables in other basins studied and even by trial and error.

After the calculation of the parameters of the selected function, it will be necessary to determine the limits of confidence and goodness of the fit tests conducted. It should be noted, that there are methods to calculate confidence limits for a given station and a given return period.

For more information on this subject, we recommend consulting the book Hydrologic Design, Fatorelli & Fernández, 2011. In this document only a few probability distributions will be listed, which are commonly used in hydrology with their respective distribution functions, in order to provide a useful reference.

a) Normal distribution o Gaussiana

[4] (Aparicio, 1989) (Fatorelli & Fernández, 2011)

It arises from the central limit theorem value, which states that a random variable x is normally distributed with average μ and standard deviation σ.

The probability distribution function gives the probability that X is less than or equal to x, as follows:

$$F(X < x) = \left(\frac{1}{\sigma.\sqrt{2\pi}}\right).\int_{-\infty}^{x} \exp\left(\frac{(x-\mu)^2}{2\,\sigma^2}\right)dx \qquad (4\text{-}30)$$

which represents the area under the curve equal to the unit value symmetrical about the average with a domain from minus infinity to plus infinity. This determines its use in hydrology, since usually the distribution of hydrologic data is not symmetrical around the average.

The analytical solution of the normal distribution function is currently unknown, so to find its solution using numerical methods and supporting tables for each value of μ and σ is required, in addition to the variable definition standardized z, expressed as:

$$z = \frac{x-\mu}{\sigma} \qquad (4\text{-}31)$$

It possesses a normal distribution with an average equal to zero and a standard deviation equal to 1.

From the above, the probability distribution function will become:

$$F(x) = F(z) = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{z} \exp\left(-\frac{z^2}{2}\right)dz \qquad (4\text{-}32)$$

This function is numerically calculated and tables exist for its solution. This is useful when there are cases where variables (especially when there are large number of data) tend to normal distribution.

Hydrologic practice indicates that the normal distribution adjusts reasonably well the extensive records of annual rainfall and annual discharges in rivers. In contrast to monthly and daily rainfall resources or flood flows, poorly fitting. The problem arises when there are negative values, however, in practice these values can be considered as zero, assuming a truncated normal distribution.

b) Two-parameter lognormal distribution

On this distribution function, the natural logarithm of the random variable is normally distributed. Therefore, its value range is positive and in the case of hydrology is an advantage over the normal distribution function.

The probability density function is:

$$f(x) = \left(\frac{1}{x\,\sigma_y\sqrt{(2\pi)}}\right)\exp\left[-\frac{1}{2}\left(\frac{y-\mu_y}{\sigma_y}\right)^2\right] \tag{4-33}$$

Where:
y, is the natural logarithm of x.
σ_y, is the standard deviation of y.
μ_y, is the average of y.
Among the limitations of this function is that the logarithms from the data must present symmetry around the average.

c) Three-parameter lognormal distribution

Its density function of x is:

$$f(x) = \frac{1}{(x-x_0)\sqrt{(2\pi)}S_y}\exp\left[-\frac{1}{2}\frac{(Ln(x-x_0)-u_y)^2}{{S_y}^2}\right] \tag{4-34}$$

For x values $>x_0$,

Where:
x_0, is the position parameter.
u_y, Scale or average parameter.
${S_y}^2$, Shape or variance parameter.

d) 2-Parameter Gamma Distribution

Its density function is:

$$f(x) = \frac{x^{\gamma-1}\exp-\dfrac{x}{\beta}}{\beta^{\gamma}\Gamma(\gamma)} \tag{4-35}$$

Valid for $0 \leq x < \infty$;

Where:
γ, is the shape parameter, so that $0 < \gamma < \infty$.
β, is the scale parameter, so that $0 < \beta < \infty$.

This distribution has important applications in hydrology, not only in frequency studies, but also in the generation of synthetic hydrograms. While studies of the Gamma function often gives similar results to the lognormal, its use is more complicated. (Fatorelli & Fernández, 2011)

e) Pearson III or Three-Parameter Distribution

Its probability density function is given by:

$$f(x) = \frac{1}{\propto_1 \ \Gamma(\beta_1)} \left\{\frac{x-\delta_1}{\propto_1}\right\}^{\beta_1 - 1} exp\left(-\frac{x-\delta_1}{\alpha_1}\right) \tag{4-36}$$

Where:
$\Gamma(\beta_1)$, is the Gamma function, and its value is derived from tables.
$\propto_1, \beta_1 \ y \ \delta_1$, are the parameters of the function that are evaluated from n measured data using the following equations:

$$\mu = \propto_1 . \beta_1 \ + \ \delta_1 \tag{4-37}$$

$$S^2 = \propto_1 . \beta_1 \tag{4-38}$$

$$\gamma = \frac{2}{\sqrt{\beta_1}} \tag{4-39}$$

Knowing that μ is the average of data, S^2 is the variance and γ its coefficient of bias; defined as:

$$\gamma = \sum_{i=1}^{n} \frac{\frac{(x_i - \mu)^3}{n}}{S^3} \tag{4-40}$$

The probability distribution function is:

$$F(x) = \frac{1}{\Gamma(\beta_1)} \int_0^x exp\left(-\left(\frac{x-\delta_1}{\delta_1}\right)\right)\left(\frac{x-\delta_1}{\delta_1}\right)^{\beta - 1} dx \tag{4-41}$$

f) Log Pearson Type III Distribution

It is a variant of the Pearson III distribution. The difference lies in the implementation of logarithms with base 10 to the sample values.

g) General Extreme Value Distribution (GEV)

Used when it is not possible to apply any of the above distributions. The cumulative distribution function is:

$$F(x) = \exp\left[-\left(1 - k \cdot \left(\frac{x-\mu}{\alpha}\right)\right)^{\frac{1}{k}}\right] \tag{4-42}$$

Where:
k, is a factor, which depends on the frequency return period to analyze and the asymmetry coefficient. Its value can be determined through tables.

For each case, there is a GEV distribution applicable, depending on the value of k, as follows:

If k = 0, it is distribution type I (Gumbel).
If k < 0, it is distribution type II (Frechet).
If k > 0, it is distribution type III (Weibull).

h) Values Distribution Type I (Gumbel o double exponential Distribution)

Its probability distribution function is:

$$F(x) = exp[-exp(-\alpha(x-\beta))] \tag{4-43}$$

The parameters α and β can be estimated as follows, for large samples:

$$\alpha = \frac{1.2825}{\sigma} \tag{4-44}$$

$$\beta = \mu - 0.45\sigma \tag{4-45}$$

And for small samples,

$$\alpha = \frac{\sigma_y}{\sigma} \tag{4-46}$$

$$\beta = \mu - \frac{u_y}{\propto} \tag{4-47}$$

Table 4-3 Values of u_y and $\boldsymbol{\sigma_y}$ (Aparicio, 1989)

n (years)	u_y	σ_y
10	0.4952	0.9496
15	0.5128	1.0206
20	0.5236	1.0628
25	0.5309	1.0914
30	0.5362	1.1124
35	0.5403	1.1285
40	0.5436	1.1413
45	0.5463	1.1518
50	0.5485	1.1607
55	0.5504	1.1682

n (years)	u_y	σ_y
60	0.5521	1.1747
65	0.5535	1.1803
70	0.5548	1.1854
75	0.5559	1.1898
80	0.5569	1.1938
85	0.5578	1.1974
90	0.5586	1.2007
95	0.5593	1.2037
100	0.5600	1.2065

The frequency coefficient k, from the Gumbel distribution type I, is expressed as:

$$k = -\frac{\sqrt{6}}{\pi}\left[\gamma + ln\left(ln\left(\frac{T_r}{T_r-1}\right)\right)\right] \tag{4-48}$$

Dónde:
Where:
γ, is Euler's constant equal to 0.57721
T_r, the return period in years.

When k=0, T_r = 2.33 years, which is the time the United States Geological Survey takes for the annual flow. (USGS, 1960)

i) Type II Distribution (Cauchy o Frechet)

Es un caso especial en el que se utilizan los logaritmos de x, el factor de frecuencia se calcula de la misma forma que en Gumbel; por tanto, para la función d la distribución tendríamos:

$$F(x) = \exp\left(-\exp\left(-y - \frac{s}{t}\right)\right) \qquad (4\text{-}49)$$

Where:

$$y = ln\, x$$

$$s = ln\, \beta$$

$$t = ln\, \alpha$$

This function is used for extreme values, should not be used to part-time series, but only for annual.

j) Type III Distribution (Weibull) (Chow, 1964)

When there is an upper limit, the cumulative probability equation is:

$$F(x) = \exp - \left[\frac{(x-E)}{\theta - E}\right]^k \qquad (4\text{-}50)$$

For x < E, in the range of -∞ < x < E, is calculated:

$$E = \beta + \frac{\alpha}{k} \qquad (4\text{-}51)$$

Where θ, is the highest expected value of E.

The following is a summary table of the probability distribution functions most frequently used in hydrology. For further detail of the use of the functions, consult the book Hydrologic Design from Fatorelli & Fernández, 2011:

Table 4-4 Frequencies probability distribution (Fatorelli & Fernández, 2011)

TYPE OF DISTRIBUTION	USE	OBSERVATIONS
Two and Three parameters Lognormal	Continuous variables	Rainfall, annual flows. Partial Duration Series.
Two-parameter Gamma	Continuous variables	Flow frequency and rains. Generation of synthetic hydrographs.
Type I (Gumbel)	Extreme values	Extreme values of flows

Type II (Frechet)	Extreme values, limit under zero	Log-Gumbel in a special case of type II.
Type III (Weibull)	There is an upper limit (E)	Minimum values of flow rates or rainfall.
General Extreme Values (GEV)	Includes types I, II, III	Determination of the most suitable distribution.
Log Pearson III	Continuous variables	Maximum annual flows and rainfall

4.4.8 Distribution Parameters Estimation

The methods of adjustment of data for parameter estimation aim to give them a degree of confidence; It is done through analytical and graphical methods and in the case of use of software it can be used an analytical calculation with an analytic-graph result.

4.4.8.1. Analytical methods for estimating distribution parameters

a) Method of Moments (Chow et al., 1994; Yevjevich, 1972)

It is a method introduced by Pearson, in which relations are set between the N parameters of the selected distribution and the first n times, both central or around the origin, of obtaining many equations, as there are parameters to be estimated.

That is, there will be as many equations as parameters. Central moments or moments around the origin can be taken. Its use is not recommended when there are errors in the data in the tails of the distribution where the arms are long moments and errors magnified.

b) Method of minimum squares

A widely used method, can also adjust the distribution functions, flows in rivers curves (H / Q), regression equations for correlations between flow stations, rainfall intensity-duration-frequency curves adjustment, etc.

It consists in calculating a regression line that best fits the data series; at first instance, it seeks to create a straight line. Parameters α, β, and γ are calculated, what the method seeks is to minimize the sum of squared deviations of the observed values.

The adjustment of a distribution can be done, either to one of the known probability frequency distributions or any other that the empirical observation curve graph of variable values may suggest. In the case of a function:

$$y = f(x; \alpha, \beta, \gamma \ldots) \qquad (4\text{-}52)$$

The data must be adjusted by the best estimate of the parameters α, β, and γ. The method minimizes the sum of squared deviations of the observed and calculated values as follows:

$$S = \sum_{i}^{N} (y_i - y)^2 \qquad (4\text{-}53)$$

Where:
xi and yi are the coordinates of the observed data and N the sample size. The line given by the function f(x, α, β, γ ...) It should also be minimized and therefore, all the first partial derivatives with respect to α, β, and γ should be zero. From these, the equations so derived are obtained for calculating the parameters.

For a straight line of best adjustment, there will be:

$$y = \alpha \cdot x + \beta \tag{4-54}$$

For a logarithmical line, the parameters α and β are found such as:

$$\alpha = \frac{\sum_{i=1}^{N} x_i y_i - N \cdot \mu \cdot \bar{y}}{\sum_{i=1}^{N} x_i^2 - N \cdot x^2} \tag{4-55}$$

$$\beta = \bar{y} - \alpha\mu \tag{4-56}$$

The application of the method is subject to three conditions:

- The errors between the observed and calculated must be distributed symmetrically relatively.
- Errors are mutually independent of the regression line.
- The variance along the line is constant.

These conditions are rarely met in hydrology, especially the second and third therefore it is very common to use logarithms to bring the equation to a linear trendfrecuente el uso de logaritmos para adecuar la ecuación a una tendencia lineal.

c) Maximum Verisimilitude Method

By this method (Chow, et al., 1994; Yevjevich, 1972), the values of the parameters are determined as to obtain the verisimilitude function. For a given density probability function, the infinite product or verisimilitude function of a sample of N values of a continuous variable x is:

$$L = \prod_{i=1}^{N} f(x; \alpha, \beta \ldots) \tag{4-57}$$

If the variable is discrete and the cumulative probability function is: Pi (x,; α, β) the verisimilitude function is the product:

$$L = \prod_{i=1}^{N} Pi(x; \alpha, \beta) \tag{4-58}$$

As one reaches its maximum value, for certain values of α, β, ..., applies logarithms; then the equation is:

$$Ln(L) = ln \prod_{i=1}^{N} f(xi; \alpha, \beta \ldots) = \sum_{i=1}^{N} ln\,(f(xi; \alpha, \beta \ldots)) \qquad (4\text{-}59)$$

Of its partial derivatives in α, β,... equated to zero, the maximum verisimilitude function that will be as many equations as parameters to be determined are obtained.

The method works best for large samples. In this case, it provides the best estimate of the parameters, although its practical application is more complex than other methods.

4.4.8.2. Graphic Methods

They are methods are being used, graphing a variable (flow, volume, height, etc.) on the ordinate and frequency or return time on the abscissa

For annual series of extreme values or duration frequencies series, it is well suited to full scale; while for very large ranges it is acceptable to use the logarithmic scale. Once graphed, the line can be visually drawn or by or using the analytical methods above.

The following table lists the most common graphical representations and coordinate axes.

Table 4-5 Graphic types (Fatorelli & Fernández, 2011)

GRAPHIC	ORDERLY	ABSCISSA	DISTRIBUTION
Normal	Arithmetic	Probability	Normal
Log Normal	Logarithmic	Probability	Two-parameter lognormal
Gumbel	Arithmetic	Gumbel probability	Log-Gumbel
Log Gumbel	Logarithmic	Gumbel probability	Exponential
Semi log	Logarithmic	Arithmetic	Log Pearson III
Log Pearson III	Logarithmic	Pearson III probability	Double exponential
Log – Log	Logarithmic	Logarithmic	Normal

4.4.9 Goodness of fit test

It is a tool that helps determine the reliability of the data delivered through the distribution of selected probability and applied to the data.

4.4.9.1. Chi-square Test(X^2)

It can be used to verify probability distributions, whether continuous distributions with data sets expressed as absolute frequencies of class intervals or as absolute frequencies in discrete distributions. It is evaluated in the following expression:

$$X^2 = \sum_{i=1}^{N} \frac{(f_i - n \cdot p_i)^2}{n \cdot p_i} \qquad (4\text{-}60)$$

Where:

n, is the number of class intervals for discrete variables or the number of events for continuous variables,
f_i, are the absolute frequencies observed in each event (or each class interval)
p_i is the probability of events (or intervals) calculated by the equation to verify p(x, α, β, γ).

4.4.9.2. Kolmogorov-Smirnov Test (K-S)

It is used when no prior distribution parameters are verified and works with a cumulative distribution. In this method the maximum deviation between the positions of experimental graphing (Pxi) the theoretical cumulative distribution (F (x,)) is determined. If there is a sample of n data x1, x2, x3.... xn in ascending or descending order and their graphing positions given by P(xi)= m/n+1, the graphs of a preselected empirical distribution is obtained. Then, F(x) the true value of the theoretical distribution, the maximum difference is defined as:

$$D_{0=}max[F(x) - P(x_i)] \tag{4-61}$$

Where:
Do, is the value of the maximum deviation between the experimental and theoretical curve.

4.4.10 Atypical Data (outliers)

In series hydrologic data, it is common to find data of extreme values, that are far significantly from the general bias of the sample; these may be due to various causes, whether measurement errors, natural factors such as extreme meteorological events or non-meteorological events. Therefore these outliers, must be specially treated to be analyzed.
The USWR Council (1982) It establishes a method for detecting high and low, respectively questionable data; through the following equations:

$$y_a = \bar{y} + k_0 \cdot \sigma_y \tag{4-62}$$

$$y_b = \bar{y} - k_0 \cdot \sigma_y \tag{4-63}$$

Where:
$\bar{y}$, It is the average of the logarithms of the sample, including the questionable ones (logarithms), σ_y, It is the standard deviation of the logarithm of the sample and ko is obtained from the following table:

Table 4-6 Values of ko for the resolution of equations from the USWR Council (Fatorelli & Fernández, 2011)

SAMPLE SIZE, n	Ko	SAMPLE SIZE, n	ko	SAMPLE SIZE, n	ko	SAMPLE SIZE, n	ko
10	2.036	45	2.727	80	2.940	115	3.064
11	2.088	46	2.736	81	2.945	116	3.067
12	2.134	47	2.744	82	2.949	117	3.070
13	2.165	48	2.753	83	2.953	118	3.073
14	2.213	49	2.760	84	2.957	119	3.075
15	2.247	50	2.768	85	2.961	120	3.078
16	2.279	51	2.775	86	2.966	121	3.081
17	2.309	52	2.783	87	2.970	122	3.083
18	2.335	53	2.790	88	2.973	123	3.086
19	2.361	54	2.798	89	2.977	124	3.089
20	2.385	55	2.804	90	2.981	125	3.092
21	2.408	56	2.811	91	2.984	126	3.095
22	2.429	57	2.818	92	2.989	127	3.097
23	2.448	58	2.824	93	2.993	128	3.100
24	2.467	59	2.831	94	2.996	129	3.102
25	2.487	60	2.837	95	3.000	130	3.104
26	2.502	61	2.842	96	3.003	131	3.107
27	2.510	62	2.849	97	3.006	132	3.109
28	2.534	63	2.854	98	3.011	133	3.112
29	2.549	64	2.860	99	3.014	134	3.114
30	2.563	65	2.866	100	3.017	135	3.116
31	2.577	66	2.871	101	3.021	136	3.119
32	2.591	67	2.877	102	3.024	137	3.122
33	2.604	68	2.883	103	3.027	138	3.124
34	2.616	69	2.888	104	3.030	139	3.126
35	2.628	70	2.893	105	3.033	140	3.129
36	2.639	71	2.897	106	3.037	141	3.131
37	2.650	72	2.903	107	3.040	142	3.133
38	2.661	73	2.908	108	3.043	143	3.135
39	2.671	74	2.912	109	3.046	144	3.138
40	2.682	75	2.917	110	3.049	145	3.140
41	2.692	76	2.922	111	3.052	146	43.420
42	2.700	77	2.927	112	3.055	147	3.144
43	2.710	78	2.931	113	3.058	148	3.146
44	2.720	79	2.935	114	3.061	149	3.148

Test parameters of questionable data 10% level of significance for normal distribution

4.4.11 Correlations Analysis.

When there is not enough data from one point in a basin, it is possible to transfer data between two points in the same basin, as long as both hydrologic areas are homogeneous.

Correlations can be simple, if two variables between two points in a basin are associated; or more, when there are several independent variables to associate.

A regression line is the adjusted curve to all average values of and for certain x values, the lower the dispersion of data around the regression line, the better it will be; and this association is measured by the correlation coefficient (r), which is calculated as follows:

$$r = \frac{\sum_{i=1}^{N}((x_i - \mu) \cdot (y_i - \bar{y}))}{N \cdot \sigma_x \cdot \sigma_y} = \frac{\sum_{i=1}^{N}(x_i \cdot y_i - N \cdot \mu \cdot \bar{y}))}{N \cdot \sigma_x \cdot \sigma_y} \qquad (4\text{-}64)$$

Where:
N, is the number of pairs of observations (xi, yi);
μ, $\bar{y}$, σ_x, σ_y average values and standard deviations of the observed values of x and y, respectively.

The correlation coefficient is a value ranging from +1 to -1; indicating direct correlation when positive, meaning that an increase of x corresponds to an increase in y; or inverse correlation if the value of r is negative.

It is common to use the square of the value *r*, called coefficient of determination, which indicates the proportion of 100 of the variance, which is absorbed by the regression line. Is important to stress that a correlation can be established only if the number of variables (NV) is less than the number of observations (N); the difference of these is called degree of freedom (GL); because if it has an equal number of observations and variables the degree of freedom will be zero and this is called a false correlation.

For calculating a simple regression equation, are used several semi algorithmic, logarithmic, etc. functions. However when it comes to multiple correlations, using statistical software that facilitates the task, becomes necessary.

4.5 ESTIMATION DATA OF THE MAXIMUM FLOW (RAINFALL-RUNOFF RELATIONSHIP)

The aim of these methods is to determine the maximum flow at the point of exit of the basin. If there are gauging data, statistical analysis of annual maximum instantaneous flow rates can be performed for the nearest station to the point of interest and flow rates are calculated for certain return periods.

However, it is not always possible to have historical flow data; so it becomes necessary to look for alternatives that apply to data that is in our hands. Here are some methods that can be used to estimate the maximum flow.

4.5.1 Rational Method

This empirical relationship takes into account the area of the basin, the height or rainfall intensity and characteristics of the ground surface. With these data, it calculates the maximum discharge assuming that rainfall is uniform throughout the basin and the maximum discharge will occur when the entire surface is draining, in other words, runoff from the bottom, the middle part and the farthest part of the basin are accumulated at the exit, establishing the maximum amount of water volume.

Given the above, the duration of the rain will be determined over time of concentration of the basin (concept explained below), in order to maximize the design flow.

This method is often used in designing drainage and urban roads and its application is a function of the surface of the basin. The method is limited to basins with surfaces up to 20km^2 [5], depending on the conditions governing the project in each country, local regulations and the validity of the results.

The expression for the maximum flow rate is given by:

$$Q = 0.278\, CiA \tag{4-65}$$

Where:
Q, is the maximum discharge in m^3/s.
C, is the runoff coefficient, dimensionless.
i, is the intensity of the rain design in mm/h.
A, basin area, in km^2.

The runoff coefficient C is defined as the ratio of the peak rate of direct runoff and the rainfall intensity of a storm. Note that due to the variability of the intensity of a storm runoff coefficient varies with time. That is why a better definition of C is expressed as the ratio between runoff and rainfall over a period of time. Always have to keep in mind that the proportion of rain to runoff depends on the land gradient, the porosity and permeability of the soil, vegetation, groundwater table position, among the most important factors. Also, the infiltration rate decreases as the rain continues and is also influenced by the precondition of soil moisture

There are many reference tables to determine the values for runoff coefficient, which can be used as suited to the conditions of the project. To illustrate, in Table 4-7 and 4-8 can be found different values of C according to the soil characteristics and return period for urban and rural areas.

[5] Basso 1967 - 1972 (Central American Hydrometeorological Project, PHCA)

Table 4-7 *Recommended runoff coefficients to be used in the rational method (Chow, Maidment, & Mays, 1994)*

SURFACE CHARACTERISTICS	RETURN PERIOD (YEARS)						
	2	5	10	25	50	100	500
Developed areas							
Asphalt	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete / Roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Green areas (gardens, parks, etc.)							
Poor condition (covered with grass less than the 50% of the area)							
Plain, 0 – 2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2 – 7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Gradient more than 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Average condition (from 50 to 75% of the area covered in grass)							
Plain, 0 – 2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2 – 7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Gradient more than 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good condition (more than 75% of the area covered with grass)							
Plain, 0 – 2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2 – 7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Gradient more than 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Non developed areas							
Crop areas							
Plain, 0 – 2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2 – 7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Gradient more than 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Grasslands							
Plain, 0 – 2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2 – 7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Gradient more than 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forests							
Plain, 0 – 2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2 – 7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Gradient more than 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Note: The values in Table 4-7 are the standards used in the city of Austin, Texas.

Table 4-8 Runoff coefficients for undeveloped or rural areas

RUNOFF COEFFICIENTS FOR UNDEVELOPED OR RURAL AREAS (1)				
	TYPE S OF FLOWS			
	EXTREME	HIGH	NORMAL	LOW
Relief	0.28 – 0.35 Steep, rugged terrain with average gradients above 30%	0.20 – 0.28 Mountainous with average gradients from 10 to 30%	0.14 – 0.20 Wavy with average gradients from 5 to 10%	0.08 – 0.14 Relatively flat land with average gradients from 0 to 5%
Soil Infiltration	0.12 – 0.16 Inefficient soil cover, with either thin rock or soil mantle of negligible infiltration capacity	0.08 – 0.12 Slow to take water, clay or topsoil, surface soils with low infiltration capacity, imperfect or poorly drained	0.06 – 0.08 Normal, with soils from light textured soils to moderately well-drained soils, clayey sand, silt and clay silts	0.04 – 0.06 High, deep sands and other soils that store water rapidly, very well drained light soils
Vegetation Coverage	0.12 – 0.16 Inefficient, bare or sparsely plant cover	0.08 – 0.12 From poor to regular, clean crops, or poor natural cover, less than 20% of the drainage area with good cover	0.06 – 0.08 Fair to good, about 50% of the area covered with grass and forest lands, not more than 50% in areas of crop production	0.04 – 0.06 From good to excellent, about 90% of the drainage area with good grasslands, forests or groves or covered or equivalent
Sewage Surface	0.10 – 0.12 Superficial negligible depressions bare and flat; steep and short drainages, without swamps	0.08 – 0.10 Under well-defined short drainage systems without ponds or swamps	0.06 – 0.08 Normal, considerable surface depressions, lakes and ponds and swamps	0.04 – 0.06 High, high sewage surface, drainage system not abruptly defined, large floodplains and numerous ponds and swamps
Example	Given: A consistent rural basin of 1) undulating terrain with average gradients of 5%, 2) clay soil types, 3) Grasslands areas, and 4) normal surface depressions. Find: the runoff coefficient, C, for the basin above-specified		Solution Relief: 0.14 Soil infiltration: 0.08 Vegetation Coverage: 0.04 Sewage Surface: 0.06 C = 0.32	

NOTE: The values in Table 4-8 are the standards used by the California Department of Transportation in the Road Design Manual.

The rain intensity can be selected based on studies or local references, and if having IDF curves for the region, the intensity corresponding to a period of rain equal to the time of concentration of the basin should be selected for a given return period. The time of concentration, (t_c) is defined as the minimum time required for all points of a basin to contribute with runoff simultaneously to its starting point. It is determined by the time it takes to reach the exit of the basin, the water that comes from the farthest point hydrologically, and represents the moment at which the runoff flow is constant; the hydrologic point that is the farthest, is that one from which runoff takes longer to reach the exit.

It can be obtained from experimental observations or used any of the equations set out below.

The equation commonly used to determine this is that of Kirpich- Ramser:

$$t_c = 0.0195L^{0.77}S^{-0.385} \tag{4-66}$$

Also:

$$t_c = 0.0195\left(\frac{L^3}{\Delta H}\right)^{0.385} \tag{4-67}$$

Where:
t_c, is the time of concentration in minutes.
L, is the length of the main riverbed, in meters.
ΔH, is the height difference.
S, It is the average gradient of the riverbed section for further study. It is determined as:

$$S = \frac{Hmáx - Hmín}{L} \tag{4-68}$$

Where:
$H_{máx}$, is the farthest point elevation and height of the basin.
$H_{mín}$, is the elevation at the exit of the basin.
L, is the length of the main riverbed.

During the implementation of the Central American Hydrometeorological Project (PHCA), The Basso et al. formula was created, which expression is:

$$tc = 0.01026\,\frac{L^{0.77}}{S^{0.385}} \tag{4-69}$$

Where:
L, is the length of the main riverbed to the exit in meters.
S, is the average gradient of the stretch of the riverbed under study.

In the literature, there are many expressions to determine the time of concentration of the hydrographic basins that have been developed in different parts of the world. In case of not having a locally developed expression, it can adopt one developed elsewhere, taking into account the conditions under which it was developed and that best applies to the project context. In Table 4-9, some of the expressions are shown for time of concentration:

Table 4-9 Other formulas to calculate the time of concentration

NAME	FORMULA	OBSERVATIONS
Bransby Williams	$t_c = \frac{58\,L}{A^{0.1}S^{0.2}}$ L, length of the main riverbed in km A, basin area in km^2 S, gradient of the main riverbed	Suggested by The Institution of Engineers, Australia, replacing the formula of Kirpich, due to its very low values (Yen, 1992).
California Culverts Practice (1942)	$t_c = 60\left(\frac{11.9L^3}{H}\right)^{0.385}$ L, length of the main riverbed, in feet H, difference in the basin level, in feet	Developed for small mountain watersheds in California (Chow et al. 1988).
Izzard (1946)	$t_c = \frac{526.42\,b\,L^{\frac{1}{3}}}{(C.i)^{\frac{2}{3}}}$ L, length of the main riverbed, C, runoff coefficient i, rainfall intensity, mm / h The formula is applied whenever i.L<3870	Desarrollada experimentalmente en laboratorio por el Bureau of Publics Road para flujo superficial en caminos y áreas de céspedes. Small areas without a defined hydrographic system with laminar surface runoff. The coefficient b is obtained from the expression: $b = \frac{0.0000276\,i + c_r}{S^{\frac{1}{3}}}$ S, average gradient of the basin Cr, delay coefficient depending on the type of surface. The formula should be used with restrictions from areas larger than 0.04 km^2.
General rational method	$t_c = \frac{60kL}{H^{0.3}}$ L, length of the main riverbed H, difference in the basin level in m K, on riverbed roughness	Developed in the United States. It is suggested to take a value of k=1.
Federal Aviation Administration (1970)	$t_c = \frac{1.8(1.1-C)L^{0.5}}{S_a}$ L, length of the main riverbed, in feet S_a, gradient of the basin, in % C, runoff coefficient	For small basins with runoff on the ground. Applied very often in urban areas (Chow et al.1988).
Delay equation from the United States Soil Conservation Service (SCS)	$t_c = 100L^{0.8}\frac{\left(\frac{100}{CN}-9\right)^{0.7}}{1900{S_a}^{0.5}}$ L, riverbed length in feet S_a, gradient of the basin, in % CN, SCS curve number.	Originated in rural areas. Based on the relationship t_c=1.67 t_{lag} SCS recommends its use in areas smaller tan 8km^2.
Pilgrim	$t_c = 0.76\,A^{0.38}$ A, basin area, in km^2	Developed for rural basins of Australia (Pillgrim & Cordery, 1993).
Hathaway	$t_c = \frac{0.606(L.n)^{0.467}}{{S_a}^{0.234}}$ L, length of the main riverbed in km Sa, gradient of the basin in m / m n, coefficient of Manning	Developed for small basins with prevalence of excess infiltration. Calibrated to areas of 0.04 km^2, with Sa < 1% and n < 0.800 (McCuen et al. 1984)

4.5.2 Methods based on unit hydrographs (Aparicio, 1989)

The hydrograph of watercourses is a graphical representation of flow rate variations (on the scale of the ordinate) versus time (abscissa scale) at a given point of the flow, where important components that help to identify the response to a storm basin, are distinguished. The area under the curve of the graph represents the total volume of water registered gauging site.

In Fig. 4-5, the hydrograph product of an isolated rainfall is illustrated, where we can distinguish the following points:

- Starting point of the direct runoff. At this point, the water from the storm begins to register at the exit of the basin. This moment may occur immediately after the start of the storm, during the same or even a while after the end.
- Hydrograph peak. The maximum flow occurs during the storm. It represents the most important point to be identified for design purposes.
- Starting point of the curve of exhaustion. It is the point from which all runoff from the effective rainfall has passed the level of capacity.

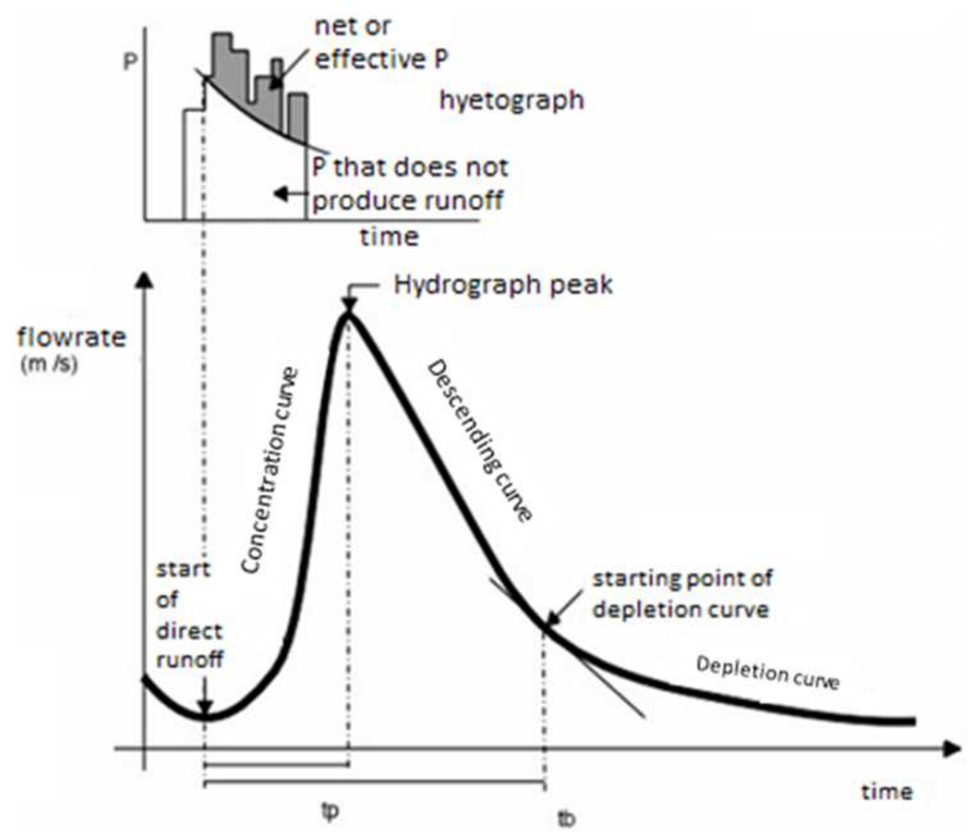


Fig. 4-5 Components of an hydrograph (Villón Béjar, 2004)

The area above the starting point of the direct runoff and the beginning of the depletion curve will be a direct runoff caused by the effective rainfall. The area below constitutes the basic flow rate caused by underground runoff.

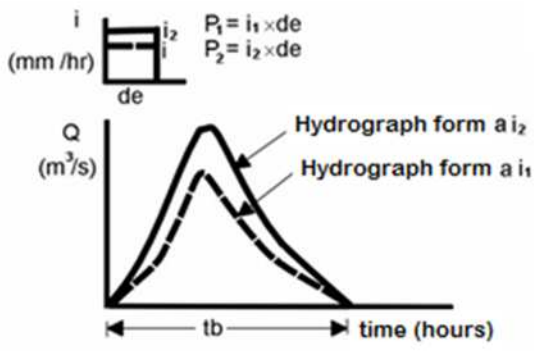
- The peak time t_p is the time from the start of direct runoff to the peak of the hydrograph.
- The base time t_b, is one that elapses from the initiation of direct runoff to the starting point of the depletion time.

There are methodologies to determine the components of a hydrograph. For further details, consult the book of Hydrology from Máximo Villón Béjar or any other book of fundamentals of hydrology.

The Unit Hydrograph (HU) of a basin is defined as the direct runoff hydrograph that results from an effective rainfall such that the total volume of excess of rainfall is the unit (generally 1mm, but may be 1 inch or 1cm), from duration d and uniformly distributed in the basin.

The method was originally developed by Sherman (1932) and is based on the following assumptions:

- Uniform distribution. Rainfall in excess has a uniform surface distribution in the basin, and throughout its duration.
- Constant time base. For a given basin the total duration of direct runoff or time base (t_b) it is the same for all storms with the same duration of effective rainfall, regardless of the total runoff volume. All unit hydrograph is linked to a duration (de) from the excess rain.



4-6 Time base constant (Villón Béjar, 2004)

- Linearity and proportionality. The coordinates of all hydrograms of direct runoff with the same time base, are directly proportional to the total volume of the direct runoff, in other words, the total volume of effective rainfall. Therefore, the ordinates of these hydrographs are proportional to each other.

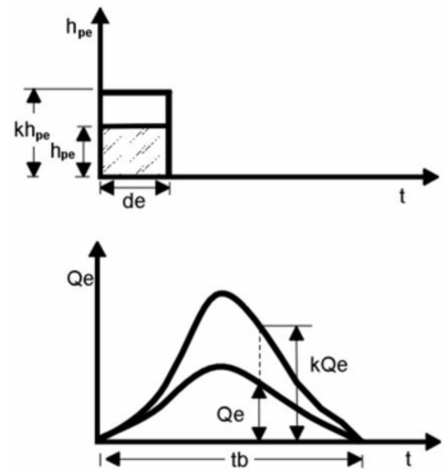


Fig. 4-7 Principle of proportionality (Villón Béjar, 2004)

- Superposition of causes and effects. The hydrograph resulting from a given period of rain may overlap hydrographs resulting from previous rainy periods.

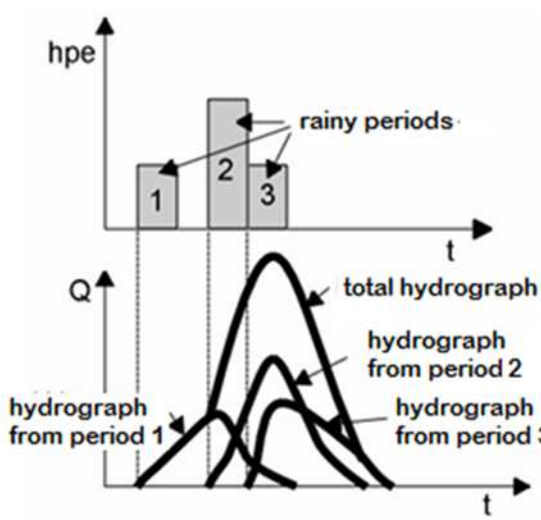


Fig. 4-8 Superposition of hydrographs (Villón Béjar, 2004)

Is worth noting that to use the method of unit hydrograph it is essential to have at least one hydrograph measured at the exit of the basin with precipitation records.

To build the unit hydrograph the following steps are employed:

- Separate the basis of direct runoff flow and determine time base.
- Get the direct runoff volume (V_e) from the storm hydrograph. Which it is the sum of direct runoff divided by the time duration in excessive from the storm.
- Get the height of rainfall in excess hp_e, dividing the volume of the direct runoff (V_e) by the area of the basin.

$$hp_e = \frac{V_e}{A} \tag{4-70}$$

- The ordinates of the unit hydrograph are obtained by dividing the ordered direct runoff by the height of rainfall excess hp_e.

The determination of the duration of excess rainfall (d) from the unit hydrograph can be done through the infiltration rate (Φ). Obtaining this rate is based on the hypothesis that the recharge of the basin from a storm under study remains constant throughout its duration. The infiltration rate units are equal to the ones of rainfall, in other words, length divided into time.

To calculate Φ, assume a rate value of infiltration and locate it on the histogram of the storm. And based on the assumption value, calculate the height of rainfall in excess (hp_e') adding increases in the ordinates of the hyetograph that are above the value of infiltration rate Φ (Fig. 4-10). If the height of the excess rainfall corresponding to the assumed value $hp_{e'}$ is equal to the value of hp_e, previously calculated, the value of Φ will be correct. If it is not, give Φ another value and repeat the procedure described above to meet the equation $hp_e' = hp_e$

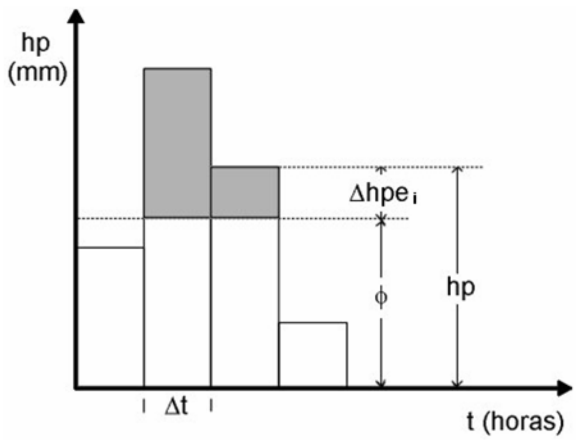


Fig. 4-9 Infiltration rate determination (Villón Béjar, 2004)

Once found the value of Φ, in the hyetograph is drawn and the excess duration is determined from the unit hydrograph. Fig. 4-10

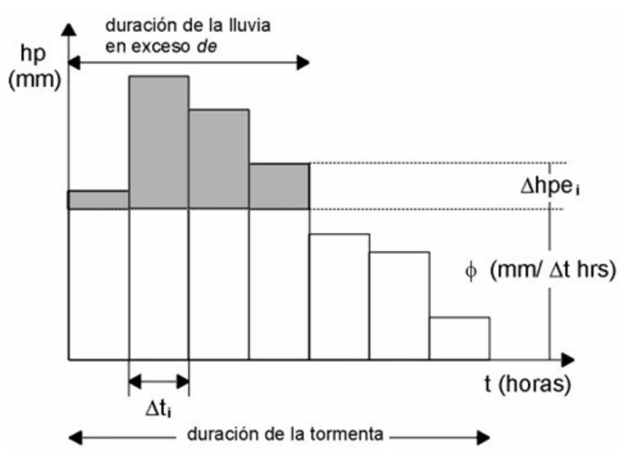


Fig. 4-10 Infiltration rate determination and excess rainfall duration (Villón Béjar, 2004)

As defined the unit hydrograph of a basin, it is possible to determine the direct runoff hydrographs for any storm with duration of excess rainfall that equals the unit hydrograph, multiplying the ordinates of the last by the value of the effective rainfall of the new storm.
In addition, due to the superposition principle of cause and effect, the unit hydrogram can be used for storm whose excess duration is a multiple of the excess duration of the unit hydrograph. Some of the references in this section can be consulted for details on the use of the unit hydrograph.

In case of not having hydrometric information or rainfall records, it is possible to create unit hydrographs based on the general characteristics of the basin. Those of this type are known as synthetic unit hydrographs and its construction follows the principle of: if the volume of surface runoff is known, the peak flow rate can be calculated assuming a certain shape of the unit hydrograph.

There are plenty of synthetic unit hydrographs. Two of the most commonly used are:

a) Triangular unit Hydrograph

Used by the United States Soil Conservation Service and developed by Mockus (1957), it provides the basic parameters of a hydrograph: peak flow (qp), time base (tb) and peak time (tp), in which the peak flow occurs. Its form shown in Fig.4-11.

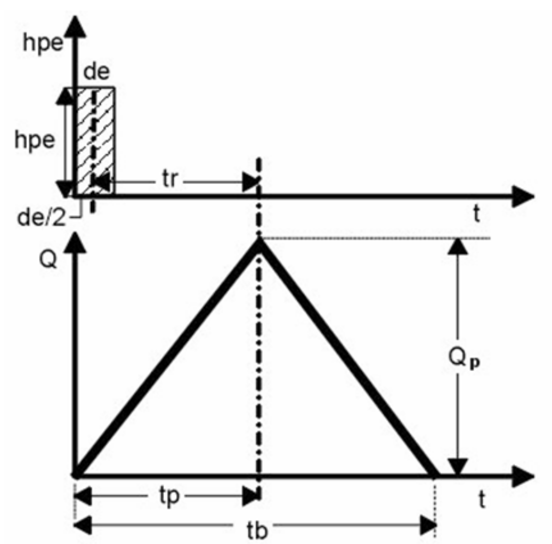


Fig. 4-11 Triangular synthetic unit hydrograph (Villón Béjar, 2004)

From the geometry of the hydrograph the expression of peak flow, is obtained as follows:

$$q_p = \frac{0.555\ \mathrm{A}}{t_b} \qquad (4\text{-}71)$$

Where:
q_p, is the peak flow in $m^3/s/mm$.
A, is the area of the basin, in km^2.
t_b, is the time base in hours.

The relation between the time base tb and the peak time tp is expressed as follows:

$$t_b = 2.67\, t_p \tag{4-72}$$

Where the peak time tp is expressed as:

$$t_p = \frac{d_e}{2} + t_r \tag{4-73}$$

being d_e the duration of excess rainfall, and t_r the delay time, which is estimated as follows:

$$t_r = 0.6\, t_c \tag{4-74}$$

where $t_{c.}$ will be the time of concentration of the basin.
Also, $t_{r,}$ can be estimated with the expression developed by Chow, in hours:

$$t_r = 0.005 \left(\frac{L}{\sqrt{S}}\right)^{0.64} \tag{4-75}$$

Where:
L, is the length of the main riverbed, in m.
S, is the gradient in %.

The duration of excess rainfall (d_e) for large basins, it can be obtained approximately through:

$$d_e = 2\sqrt{t_c} \tag{4-76}$$

For small basins we can take $d_e = t_c$, introducing the time in hours.

Ultimately, the peak flow versus time expressed basis is as follows:

$$Qp = \frac{2.08\, A}{t_p} \tag{4-77}$$

Where t_p will be given by:

$$t_p = \sqrt{t_c} + 0.6 t_c \tag{4-78}$$

The Roads Manual in Peru recommends this method in basins not larger than 30km²; although other references limit it to higher values. Also, in the same manual, it is recommended that for urban basins where t_p and t_c decrease by the sealing and channeling, assess whether it is necessary to apply the factors f_1 and f_2 to the time peak t_p to calculate a modified time peak t'_p, as follows:

$$t'_p = t_p . f_1 f_2 \tag{4-79}$$

$$f_1 = 1 - M_a k \tag{4-80}$$

$$f_2 = 1 - M_c k \tag{4-81}$$

Where:
M_a, is the percentage increase in waterproof areas.
M_c, is the percentage of channeled areas.

$$k = (-0.02185CN^3 - 0.4298CN^2 + 355CN - 6789) * 10^{-6} \tag{4-82}$$

With CN equal to the number of CSC curve. (See section 4.5.3 for its determination)

b) Snyder synthetic unit Hydrograph (Chow, Maidment, & Mays, 1994)

Snyder (1938) defined the standard unit hydrograph as one whose duration of rain t_r is related to the delay of the basin t_p, by:

$$t_p = 5.5\, t_r \tag{4-83}$$

The delay of the basin, in hours, is given by:

$$t_p = 0.75\, C_t (L * L_c)^{0.3} \tag{4-84}$$

Where:
C_t, is an empirical coefficient that depends on the characteristics of the basin. An equation proposed by Chow (1964) to obtain it:

$$C_t = \frac{0.6}{\sqrt{S}} \tag{4-85}$$

With S, as the average gradient of the basin. Snyder proposes values between 1.8 and 2.2, being the lower values corresponding to basins with higher gradients

L, is the length of the main course at km from the exit of the basin to the upstream boundary.
L_c, is the length of the main course from the exit of the basin to the point of the closest flow to the centroid of the basin area in km.

The peak flow rate per unit area of drainage $m^3/s. km^2$ from the standard unit hydrograph is:

$$q_p = \frac{2.75C_p}{t_p} \quad (4\text{-}86)$$

Being C_p the empirical coefficient of retention and sewage, varies between 0.4 and 0.9
From a unit hydrograph deducted in the basin, values from its effective duration are obtained $t_{R,}$ in hours, its delay time in the basin t_{pR} in hours, and its peak flow per unit of drainage area, q_{pR} in $m^3/s.\ km^2$.

The standard delay of the basin is:

$$t_p = t_{pR} + \frac{t_r - t_R}{4} \quad (4\text{-}87)$$

The ratio between q_p and the peak flow rate per unit drainage area q_{pR} the unit hydrograph required is:

$$q_{pR} = \frac{q_p t_p}{t_{Pr}} \quad (4\text{-}88)$$

The expressions for estimating the flow rate were obtained from study of basins located in the Appalachian Mountains of the United States with surfaces ranging from 30 to 30,000km^2.

When it comes to work in basins with little hydrological information, it is not justified to make complicated analysis for the calculation of the synthetic unit hydrograph to estimate the maximum flow. For this reason, it is best to use simple and easy to apply methods. The synthetic triangular hydrograph meets the above characteristics, and the advantage over Snyder in its application is that it does not depend on the determination of coefficients depending on the characteristics of the basin studied.

4.5.3 Flood routing

The Routing is the process by which the evolution of a hydrograph is known, to the extent that it runs along a riverbed through a canal or a reservoir. This calculation is of great importance in the hydrologic analysis because it allows estimating the change in the value of the maximum flow as it moves downstream from the water flow.

There are various methods to evaluate Hydrologic Routing, which are grouped into hydrologic methods, based on the continuity equation, and hydraulic methods, in addition to the continuity equation; the equations of motion fluid are used.

One of the hydrologic methods mostly used for its simplicity, is the Muskingum. Which states that sewage in the section of a riverbed can be decomposed into two parts: prism storage and wedge storage. As shown in Fig. 4-12.

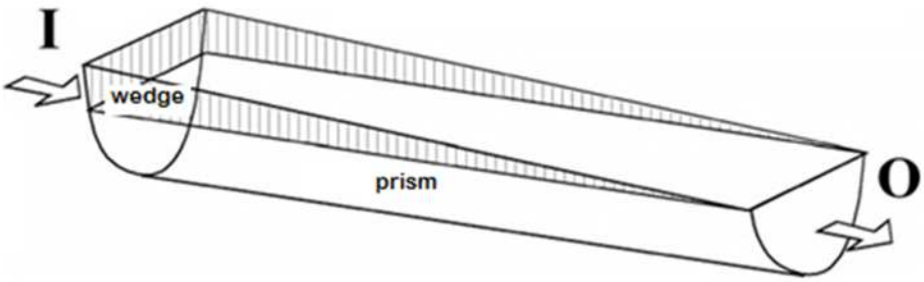


Fig. 4-12 Prism sewage and wedge sewage. Muskingum method. (Sánchez San Román, 2012)

The prism storage is expressed as follows:

$$S_{prism} = K.O \quad \text{(4-89)}$$

And the wedge storage, as:

$$S_{wedge} = K.X.(I - O) \quad \text{(4-90)}$$

From sum of the two expressions is obtained:

$$S = K\,[X.I + O(1 - X)] \quad \text{(4-91)}$$

Where:
S, is storage for the section considered a riverbed.
I, inflow in a riverbed stretch.
O, outflow on the stretch of riverbed.
K and X, constant for the stretch of riverbed.

Applying the continuity equation for two consecutive times t_{i-1} and t_i, separated, an intervalΔt, an exit flow rate is obtained at t_i time, as follows:

$$O_i = I_i \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} + I_{i-1} \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} + O_{i-1} \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \quad \text{(4-92)}$$

which can also be expressed as:

$$O_i = C_0 I_i + C_1 I_{i-1} + C_2 O_{i-1} \quad \text{(4-93)}$$

K and X are constants that depend on each stretch of the riverbed and represent the damping hydrograph throughout the stretch of the riverbed. The first can be likened to the running time of the kinematic wave and the second can be between 0 and 0.5, but is normally taken as 0.2.

For more details on Routing and the different calculation methods, Applied Hydrology from Chow, Maidment, & Mays, 1994 can be consulted.

4.5.4 Method from the United States Soil Conservation Service (SCS), TR-55 for the effective rainfall calculation

This method was developed by the United States Soil Conservation Service (SCS), is commonly used to determine the direct surface runoff in road engineering. It is also known as TR-55 and can be used to estimate direct runoff volumes and peak rate of discharge.

The basic premise used for the development of this method is that the depth of the layer of direct runoff or rainfall excess P_e depends on the height of precipitation P. Part of the rain that falls at the beginning of a storm, known as initial abstraction (I_a), will not be part of the direct runoff.

Maximum retention potential S from the ground surface (similar to the runoff coefficient C concept in the rational method) is a measure of the impermeability of the basin area.
The method consists of two parts: the first, the direct runoff or effective rainfall is determined. The second part considers the peak or maximum discharge using the value of P_e, initially obtained.

The expression defined by the SCS to determine P_e is as follows:

$$P_e = \frac{(P - I_a)^2}{(P - I_a) + S} = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (4\text{-}94)$$

Where:
P_e, is the runoff or excess rainfall in inches.[6]
P, the total rainfall in inches.
S, maximum retention potential after the start of direct runoff, in inches.
I_a, It is the initial abstraction, including surface sewage, interception and infiltration prior to direct runoff. The relationship between I_a and S, empirically developed from basins data is $I_a = 0.2S$

To determine P_e, firs S has to be calculated in inches, which is determined as:

$$S = \frac{1000}{CN} - 10 \qquad (4\text{-}95)$$

The dimensionless number CN is the number of the direct runoff curve. Its value varies from 0, for a permeable surface, all the way to 100 for completely impermeable surfaces and water surfaces. For natural CN surfaces < 100.

The number CN considers the characteristics of the basin, such as soil type, land use, hydrologic condition of the cover and the initial soil moisture just before the storm (antecedent soil moisture). Some CN values are shown in Table 4-10:

[6] 1 inch = 25.4 mm

Table 4-10 Runoff curve numbers for selected uses of agricultural, suburban and urban land and antecedents moisture conditions AMC II. I_a=0.2 S. (Chow, Maidment, & Mays, 1994)

LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP			
		A	B	C	D
Cultivated Land[1]:	Without conservation treatment	72	81	88	91
	With conservation treatment	62	71	78	81
Pastizales:	Poor Conditions	68	79	86	89
	Optimal Conditions	39	61	74	80
Vegas de ríos:	Optimal Conditions	30	58	71	78
Forests:	Thin trunks, poor cover, no herbs,	45	66	77	83
	Good cover[2]	25	55	70	77
Open areas, grass lands, parks, golf courses, cemeteries, etc.					
Optimal conditions:	grass cover 75% or more	39	61	74	80
Acceptable Conditions:	Cover grass at 50 to 75%	49	69	79	84
Commercial business areas (85% waterproof)		89	92	94	95
Industrial districts (72% waterproof)		81	88	91	93
Residential[3]:					
Average lot size	Average waterproof percentage[4]				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, entrances, etc.[5]		98	98	98	98
Roads and Highways:					
Paved with gutters and culverts[5]		98	98	98	98
Gravel		76	85	89	91
Land		72	82	87	89

1. For a more detailed description of the curve numbers for agricultural land uses, refer to Soil Conservation Service, 1972, Chapter. 9.
2. A good cover is protected from grassland, and waste removal of ground cover.
3. The numbers of the curve are calculated assuming that runoff from houses and an entrance goes to the street, with a minimum of water from the roof facing the lawn where additional infiltration can occur.
4. The remaining permeable areas (lawn) are considered as grassland in good condition for these curve numbers.
5. In some countries with warmer climates, 95 can be used as curve number

This table is applicable for normal Antecedent Moisture Conditions AMC (AMC II). To dry conditions (AMC I) or humid conditions (AMC III), the equivalent curve numbers can be calculated by:

$$CN(I) = \frac{4.2\, CN(II)}{10 - 0.058\, CN(II)} \tag{4-96}$$

$$CN\,(III) = \frac{23\, CN\,(II)}{10 + 0.13\, CN(II)} \tag{4-97}$$

The effect of the Antecedent Moisture Conditions in direct runoff is considered by soil classification into three categories:

- Condition I (AMC I): dry soil but not to the point of dryness, satisfactory crops can be performed.
- Condition II (AMC II): average condition or normal conditions.
- Condition III (AMC III): In the last five days, it has presented intense rainfall, or light rainfall on low temperatures, saturated soils.

The limits of rainfall, presented as guidelines for determining the Antecedents Moisture Conditions (AMC), are presented in Table 4-11

Table 4-11 Classification of types of moisture antecedents (AMC) for the method of abstractions rain from SCS

GROUP AMC	TOTAL ANTECEDENT RAIN IN 5 DAYS (INCHES)	
	INACTIVE STATION	GROWING STATION
I	Under 0.5	Under 1.4
II	From 0.5 to 1.1	From 1.4 to 2.1
III	Over 1.1	Over 2.1

CN values recommended by the Soil Conservation Service have been defined based on the type of soil and the use given. Therefore, there have been defined four groups of soil and their characteristics are described below:

Group A: deep sand, deep soil deposited by wind, added silts.

Group B: shallow soils deposited by the wind, sandy loam soil.

Group C: clayey loams, shallow sandy loam soils, soils with low organic content and soil with high clay content.

Group D: soils significantly expanded when wet, highly plastic clays and certain saline soils.

To a basin with various soil types and for various uses, one can calculate a multiplied CN.

The graphical representation of the relationship between P and P_e for many basins, it was determined by SCS and is shown in Fig. 4-13.

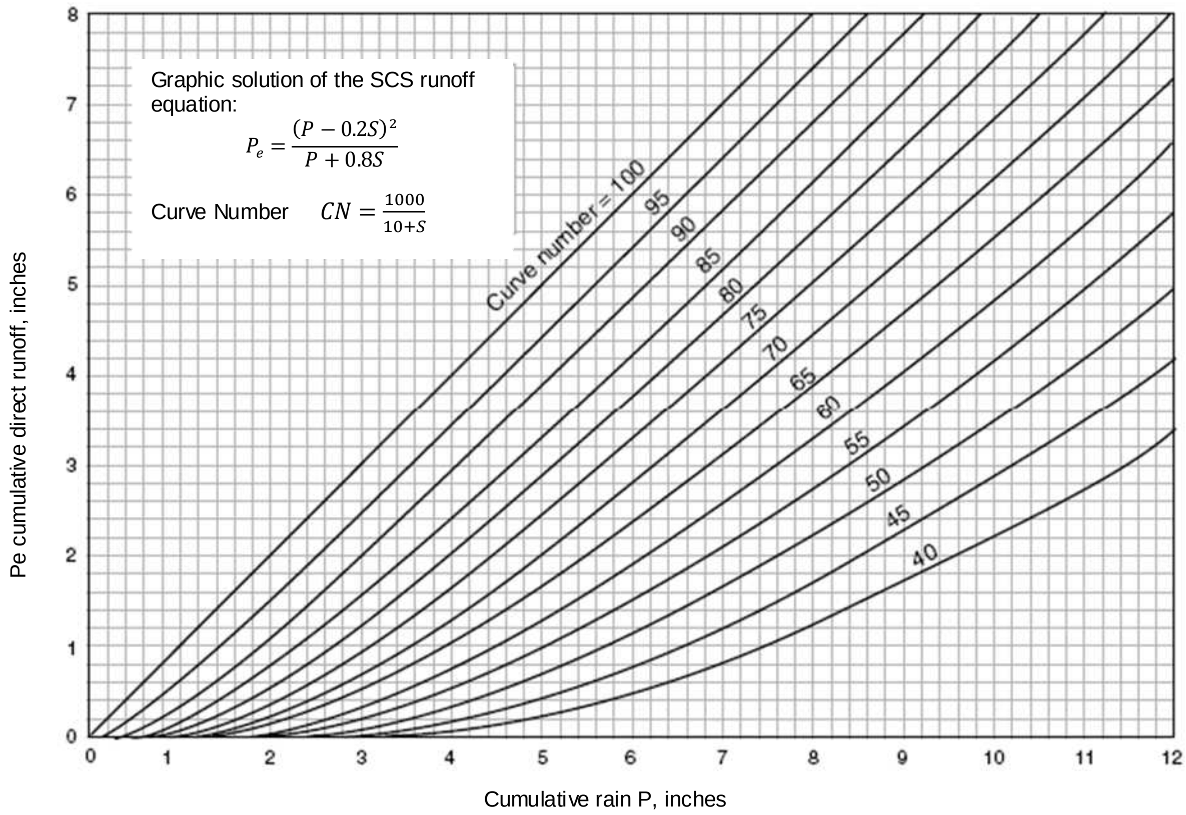


Fig. 4-13 Runoff equations solution from SCS (Source: Soil Conservation Service, 1972)

For the calculation of peak flow Q_p, an alternative is to use the triangular hydrograph developed by Víctor Mochus (1967), from the United States Soil Conservation Service.

4.5.5 Methods based on direct flow measurement [7]

When there are flow rate records, it is possible to use a simple empirical method, called Distribution Graph (Bernard, M.M., 1935) that depending on various hydrographs recorded and a frequency study of maximum flow rates only; no rainfall data are used. The value of flow rate and volume is maximized in a hydrograph project that is in a way the envelope of the hydrograph registered.

The methodology is as follows:

- 3 or 4 maximum high waters are selected
- The base flow is separated into all of them.
- It is calculated for each the total volume of direct runoff.
- For each high waters, the percentage graph of distribution is constructed, that for each time interval (Dt) expresses the volume percentage of runoff relative to the total volume.
- From all, that in which the percentage of peak volume is lowest relative to the total volume is selected.

[7] (Fatorelli & Fernández, 2011)

With that percentage distribution of volumes, the project hydrograph is reconstructed based on a peak flow rate that becomes equal to the return period flow obtained from statistical analysis of flow frequency. Getting the other flows in percentage terms with the selected hydrograph.

4.5.6 Data transference[8]

If the hydrometric station on the flow of water under study is not exactly at the project site of the road drainage work, but within the same basin, it is possible to transfer annual maximum instantaneous flows of different return periods of this station to the site of the project through area drainage relations, as follows:

$$Q_{sp} = Q_{EH}(\frac{A_{sp}}{A_{EH}})^X \qquad (4\text{-}98)$$

Where:
Q_{SP}, is the flow rate at the project site, in m^3/s.
Q_{EH}, is the flow rate in the hydrometric station, in m^3/s.
A_{sp}, is the area of the hydrographic basin to the project site, in km^2.
A_{EH}, It is the area of the hydrographic basin to the hydrometric station, in km^2.

The exponent x is a value, which usually ranges from 0.5 and 0.75. As a lack of research data, it is customary to take a value equal to 0.5.

The methodology is applicable in basins where the area to be transferred is maintained between the upper and lower limits of 50% of the original drainage area.

4.6 CONSIDERATIONS ON IT TOOLS OF HYDROLOGIC ANALYSIS

It is increasingly common to use computer models to generate road hydrographs. It should be recognized that these models solve empirical formulas, in some cases, or use simulation techniques. The simulation is based on the division of the basin into smaller areas to which a discretization of the design storm is applied and the volume is subtracted due to infiltration and interception losses. The remaining rain is simulated using a surface flow routine.

The total response of the drainage area will result from the sum of the surface flows of the different subareas in which the original surface was divided.

Is worth mentioning that the validity of the results of resolving an empirical formula, but above all, of simulation models, is increased through the use of measured historical data, which will be used to calibrate the model parameters. That is why one of the disadvantages of using simulation models is that they require a large amount of initial data and a wide user experience for reliable results.

[8] (Fatorelli & Fernández, 2011)

There is a variety of tools for hydrological modeling. FHWA and AASHTO have created a package for personal computers called HYDRAIN, consisting of several programs. The available information can be consulted in the following website:
http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm

Other program for hydrologic modeling is HEC-HMS developed by the United States Army Corps of Engineers and is used to simulate the hydrologic processes in basins. The software includes traditional hydrologic analysis procedures, as events of infiltration, unit hydrographs and hydrologic routing. HEC-HMS also includes modules for evapotranspiration, and calculation of soil moisture. It can be downloaded free from the United States Army Corps of Engineers website:
http://www.hec.usace.army.mil/software/hec-hms

Rainfall-Runoff Modeling System (PRMS) is a set of modules that represents the physical processes of a river basin. It was developed by the United States Geological Service (USGS) to assess the effects of various combinations of geomorphology, type and use of land, vegetation and climatic parameters on the hydrologic response of the basin. The download site is:
http://wwwbrr.cr.usgs.gov/projects/SW_MoWS/PRMS.html

TETIS is a hydrologic simulation model and from the type sediments cycle distributed in space by a subdivision of the basin in regular cells, based on physical parameters. It is a global model, which means that with the same model problems from both, flood and erosion as well as water resources problems can be solved. Besides, it has a powerful algorithm for automatic calibration of the effective parameters and the initial values of all state variables, which greatly facilitates its practical implementation. Website:
http://lluvia.dihma.upv.es/ES/software/software.html

Finally, regardless of the methodology used, never forget that the final result of the hydrologic study must provide the value of maximum flow rate for the return period corresponding to the drainage structure design, in order for the hydraulic analyst can evaluate the behavior of a structure for this value.

4.7 HYDROLOGIC STUDY

Below as a guide in the hydrologic analysis, the minimum content that should have the studies to be presented, are recommended; considering that there will be peculiarities for each country in the specific structure for each area presented on the recommendation of content:

- Gathering information.
- Relevance and justification of the used return periods.
- Study and determination of rain analyzes.
- Characterization of the basin.
- Analysis and calculation of flows for different return periods.
- Final conclusions and recommendations of the study.
- Appendixes.

4.7.1 Example of hydrologic studies for drainage works

As an illustration, an example of the requirements made by the Public Work Planning Department of from the Ministry of Public Works of El Salvador is shown, for the presentation of hydrologic studies for road projects:

The hydrologic study will aim to analyze the rainfall regime and other hydrologic characteristics of the area covered by the project and the basins affected by the trace, in order to determine the flows generated by these and properly size the necessary drainage works.

All values obtained must be clearly justified from some initial data, defining the process used to define these values and summarizing these in final tables, so that they are sufficiently clear, both the results and the process followed.

4.7.1.1. Climatology

The existing publications of the Division of Meteorology and Hydrology of the Department of Renewable Natural Resources, Ministry of Agriculture, or any other body will be consulted, in relation to climate zone data. If the work is located in a place where some of the data collected in these publications are not representative because they are based in remote weather stations of the trace, a specific study based on data from the Division of Meteorology and Hydrology will be made available.

It is mandatory the incorporation of the original data supplied and the process followed for selection, in which will be considered the trace proximity conditions, number of years with complete data and altitude of the station being recorded.

An overview of the selected stations will be made, expressly stating the identification code, hydrographic basin in which the type of station is located (pluviometry, thermos-pluviometry, etc.), name, coordinates, altitude, number of years of data and number of years with complete data.

On a plane on an appropriate scale, and in no more than two pages, the position of the selected stations will be covered, indicating its name and code, as well as the trace object of the project.

4.7.1.2. Minimum contents

The study will be divided into three sections:

- Obtaining, by statistical analysis, major climatic variables
- Classification and climate rates
- Determining the number of usable days in the implementation of the works

Within the section of climate variables, the following are obtained:

a) Rainfalls:

- Monthly and annual average rainfall

- Maximum rainfall in 24 hours (by month and year)
- Maximum monthly rainfall
- Number of rainy days
- Number of days of storm

b) Temperatures:

- Monthly and annual average temperature
- Average minimum temperature (monthly and annual)
- Average maximum temperature (monthly and annual)
- Absolute minimum temperature (monthly and annual)
- Absolute maximum temperature (monthly and annual)
- Maximum oscillation of temperatures

c) Other information of interest:

- Average relative moisture
- Average daily evaporation
- Average annual number of sunny days
- Average annual number of clear days
- Analysis of the prevailing winds (direction, route, velocity, etc.)

Wherever possible, the results are presented as graphs with the specification of the representative values.

In the section on climate rates, diagrams from each of the selected stations which are reflected the dry and periods over the year, are incorporated.

To study the usable days in the implementation of the main units of work to be continued, the values previously obtained will be taken into account, setting the number of usable days throughout the year and giving its months where the different units of major works of the road can be performed, as follows:

- Soil removal
- Asphaltic concrete works.
- Hydraulic concrete works.
- Complementary works.

4.7.1.3. Hydrology

This section of the project will start with an overview of the hydrology of the area based on the available data of the geology of the area and visits made to the trace, specifying crossed watercourses, upwelling, streams, estuaries, swamps, wells, etc. located in the area of the project and that directly or indirectly affect the trace. This description will serve as a basis for estimating the studies that will be further developed and the necessary data to be collected for it.

In addition to the pluviometry data of the Division of Meteorology and Hydrology, which should have the same treatment described for climate data, the necessary contacts with the organizations concerned (regents Organizations, rivers, reservoirs, coasts, ports, etc.) should be maintained to receive any additional information available such as gauging water courses, tidal routes of maximum flood levels in reservoirs, as well as factors affecting the subsequent design of the drainage works necessary, or interference with other projects in development.

a) Study of the expected máximum rainfall

Based on the maximum daily rainfall data obtained in the previous section, frequency graphs of maximum rainfall will be made in different months of the year for each selected station.

In 24 hours, the expected maximum rainfall will be calculated for return periods of 5, 10, 25, 50, 100 and 500 years. For that, data collected at selected meteorological stations will be used, generating the series of maximum rainfall in 24 hours, indicating the year and month of occurrence, on which Gumbel distributions will be applied.

It will be done a summary table with the treated maximum rainfall stations adopted in them for the different return periods.

b) Intensity-Duration curves determination

The intensity-duration curves for different return periods specified above will be defined, so that when coming with a specific duration for the rain intensity, it will define us the time needed to calculate the flow of the basin.

c) Basins study

The various basins sloped to the trace over scale plans 1:1000, 1:5000 and smaller scales needed so that they can reflect the limits of the large basins. These plans will depend on features and curves of sufficient level so that the correct layout of the partition is observed.

From each basin it will be obtained physical characteristics needed for calculating flows generated therein, carried out summary tables necessary where specified, at least the following characteristics of each basin:

- Nomenclature.
- Expected drainage work.
- Basin area to the point of intersection with the trace.
- Length of the basin following the possible route of runoff.
- Gap between the head of the basin and the point of impact on the trace.
- Average resulting gradient.

Different land uses, specifying its incidence in the total basin.

d) Concentration periods

Times of concentration of each of the basins will be determined, specifying them in a table, where the values of the basins described above include are included.

e) Runoff Coefficients

Each basin runoff will be determined, depending on the vegetation, type of crop and soil type of the same.

f) Flow rates calculation

For the calculation the flow rates generated by the basins proven methods will be followed.

Therefore, to calculate maximum flows in natural basins, with an area of less than 50 km2, it can be used the Rational Method, while for larger areas methods such as the Unitary Triangle Methods, Isochrones, etc. will be applied.

When using IT applications must be included a summary of the calculation procedure performed by the application, and a description and analysis of the parameters used in the process.

Once calculated flow rates of the various basins, a summary table will be drawn up specifying:

- Name of the basin.
- Expected drainage work.
- Basin area to the point of intersection with the trace.
- Time of concentration.
- Maximum hourly intensity.
- Average resulting runoff coefficients.
- Flows for return periods of 5, 10, 25, 50, 100 and 500 years.

g) Others Necessary Studies

Depending on the particular characteristics of the trace,it will be necessary to have studies or specification of particular data of tidal flows, flow rates spillways in dams, water levels in reservoirs, gauging rivers and estimation of maximum flows in them, gauging springs and upwelling which must be conducted in accordance with the Companies or Competent Bodies in each case.

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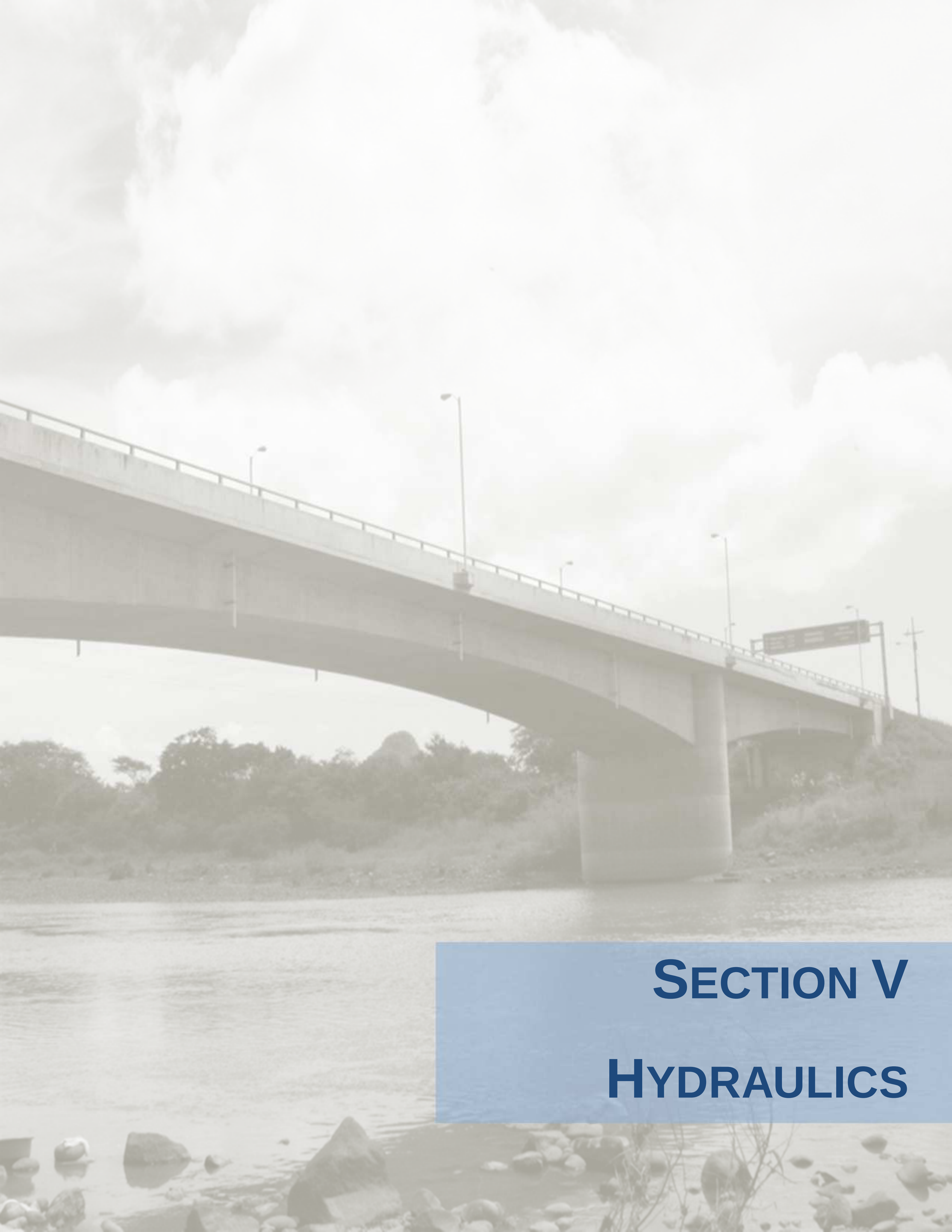


SECTION V

HYDRAULICS

HYDRAULICS

5.1. INITIAL CONSIDERATIONS

Among the works that have to be done for the successful implementation of a road project is the design of the works required for the evacuation of water that can affect the proper performance of the road, both the durability of the materials, and operational problems and even interruptions in the use that is given to it.

Therefore, it is important to consider the most relevant aspects for the development of these drainage works, from the stage of planning to the execution of the work.

Some of the aspects to consider are:

- Location, importance and magnitude of the road project.
- Location of the drainage works.
- Amount of flow to drain (hydrology of the site).
- Hydraulic capacity of the work proposed.
- Initial site conditions and possible effects when making the channeling flow through the drainage work.

Because this document aims to serve as a guide for consideration of these issues, and specifically in this section to hydraulic, in the following paragraphs, there will be a brief description of the most important components of these.

5.2. COMMON TYPES OF DRAINAGES IN ROAD

Drainage works on roads can be classified into two areas, which are the surface drainage (lengthwise and cross) and subsurface drainage (which in some cases are also called underground drains or sub-drains); the following is a list of the most common work, according to this classification:

5.2.1. Lengthwise drainage

The following works are known: curbs, gutters and under gutters and spillways or lowered[9]. As mentioned earlier in this guide calculation methodologies of some of the works will be developed, which make up the drainage on the road, turning to the case of lengthwise drainage, a methodology for calculating gutters, one presented below.

Gutters are drainage structures that capture water from surface runoff from the platform of the road and the cut slopes, lengthwise leading them to ensure its proper disposition (National Road Institute, 2009).

The ones built in landfill areas also protect the edges of the berm and the landfill batters from erosion caused by rainwater, as well as serving on many occasions, to continue with the cutting gutters to a natural stream in which it can be delivered.

[9] Also known in the region as energy sinks or downspouts

In cutting areas, disposition points are sewage collecting boxes and side exits to natural terrain in a change of cut landfill. In landfill, water is disposed to natural terrain by downspouts or reliefs and in the gutters of a central water separator; they are also conveyed to the collection box of a culvert.

The gutters must be located in essentially all cuts, in those landfills susceptible to erosion and all internal margin of a separator that receives rainwater from roads.

The abscissa in which must it be placed gutters and drainage points and must be obtained from the analysis of the profiles of the road (with its lines of bevel cut and fill) and the camber diagram where the sense of pumping (transversal gradient) in the case of divided highways indicated (Instituto Nacional de Vias, 2009).

For the design of these works is both a methodology developed through the use of nomogram, as through the development of formulas of IZZARD.

The capacity of a gutter depends on its shape, gradient and roughness. If the cross and lengthwise gradients from the road is known, the gutter can be represented as an open canal of triangular section and its hydraulic capacity can be estimated with the formula of uniform flow from Manning. This has been usually represented by the nomogram of IZZARD, which solves the following equation:

$$Q_0 = 0.375 * \sqrt{l} * \left(\frac{z}{n}\right) * y_0^{\frac{8}{3}} \tag{5-1}$$

Where:
Q_O, flow rate in the gutter, in m^3/s.
I, lengthwise gradient.
1/z, cross gradient.
n, roughness coefficient of Manning.
y_O, flow depth in m.

With regard to the values of n is to be taken into account that these are to be considered with its increased depending on the characteristics of the material of the gutter, so in the table below some characteristic values of n are presented.

Table 5-1 Roughness coefficient of Manning (Chow, 2004)

TYPE OF SURFACE	"n"
Gutter of concrete with smooth finish	0.012
Asphalt coating with smooth texture	0.013
Asphalt coating with rough texture	0.016
Cement slurry coating a) Finished with frotachado b) Smooth finish by hand c) Rough finish by hand	 0.014 0.016 0.020
Coating with cobblestones	0.020
Gutters with lengthwise small gradients (up to 2 %) subject to the accumulation of sediments, the "n" values indicated must be increased by + 0002 to 0.005	n

When the cross section of the gutter consists essentially of a pavement with uniform gradient, el caudal It can be quickly calculated using the nomogram of Izzard for runoff in a triangular canal, which is shown in Fig. 5.1 (Note: the reference of the nomogram, this system is in English, so that when the calculations should be taken into account), besides, calculations for geometric parameters for this type of canal are in Table 5-2.

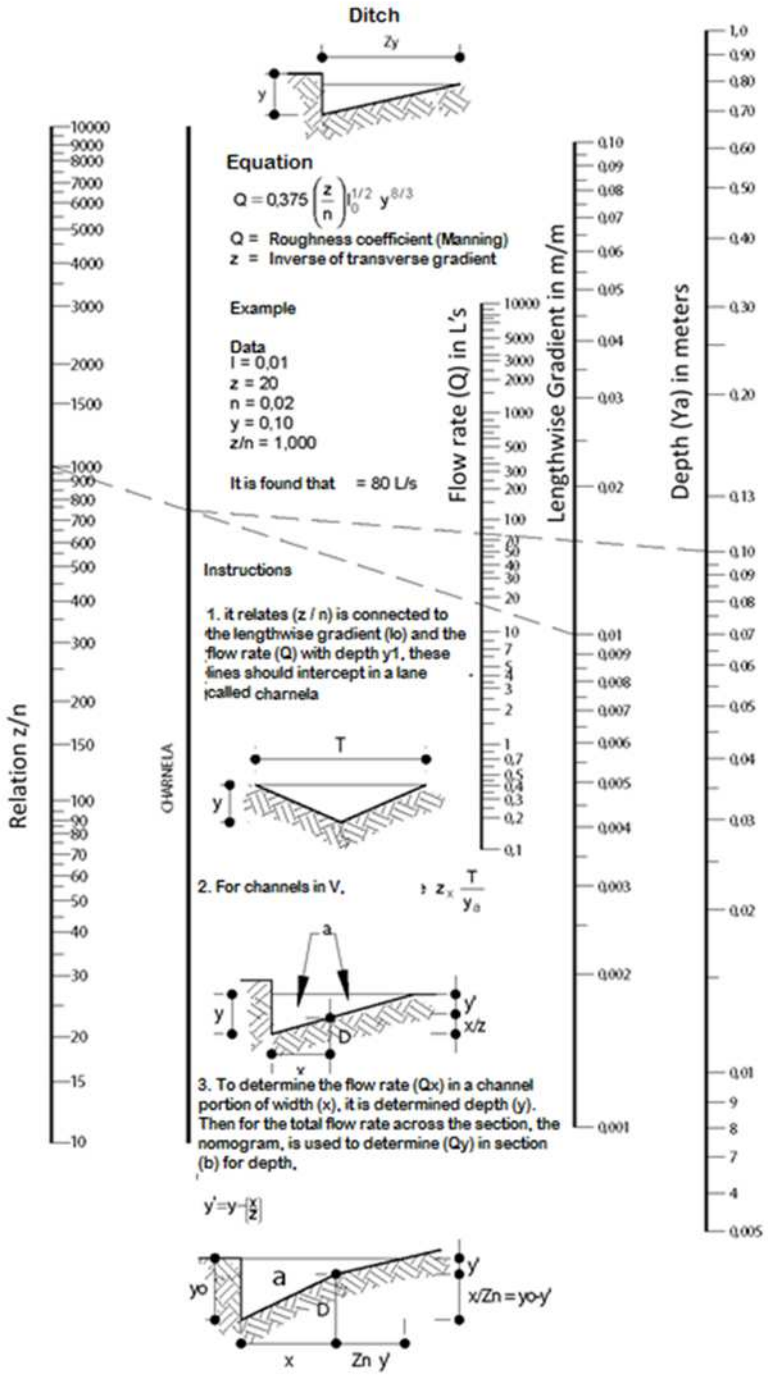
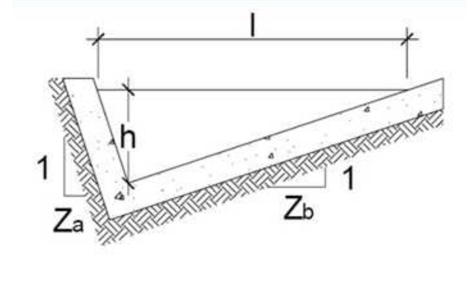
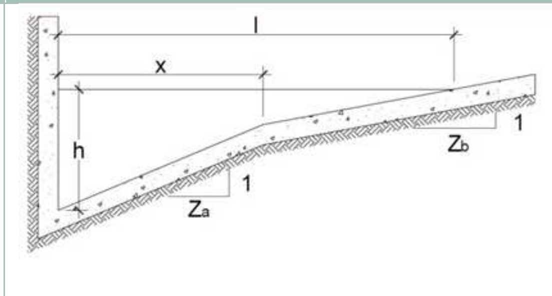


Fig. 5-1 Nomogram of IZZARD for calculating ditches or canals. (Bolivian Institute of Standardization and Quality, 2007)

Table 5-2 Hydraulic capacity of gutters and triangular canals (MTI, 2008)

	SIMPLE TRIANGULAR CANAL	COMPOSITE TRIANGULAR CANAL
		
Surface width (l)	$(z_a + z_b)h$	$x + z_b\left(h - \frac{x}{z_a}\right)$
Area (A)	$\frac{(z_a + z_b)h^2}{2}$	$xh + \frac{z_b h^2}{2} + \frac{x^2}{2z_a}\left(\frac{z_b}{z_a} - \frac{2z_b h}{x} - 1\right)$
Wet perimeter (P_m)	$\left(\sqrt{1 + z_a^2} + \sqrt{1 + z_b^2}\right)h$	$h + \sqrt{x^2\left(1 + \frac{1}{z_a^2}\right)} + \sqrt{z_b^2 + 1}\left(h - \frac{x}{z_a}\right)$
Hydraulic Ratio (R_h)	$\frac{(z_a + z_b)h}{2\left(\sqrt{1 + z_a^2} + \sqrt{1 + z_b^2}\right)}$	$\frac{xh + \frac{z_b h^2}{2} + \frac{x^2}{2z_a}\left(\frac{z_b}{z_a} - \frac{2z_b h}{x} - 1\right)}{h + \sqrt{x^2\left(1 + \frac{1}{z_a^2}\right)} + \sqrt{z_b^2 + 1}\left(h - \frac{x}{z_a}\right)}$

Another way to estimate the hydrologic flow rate is to use the rational formula (developed in previous sections) as contribution areas for the gutters or canals are small, besides it can also be used the method IZZARD, applicable to areas with dispersed runoff without riverbeds defined.

The specific flow is the sum of the volumes produced by the gutter, the road and batter, according to what is shown in Fig.5-2

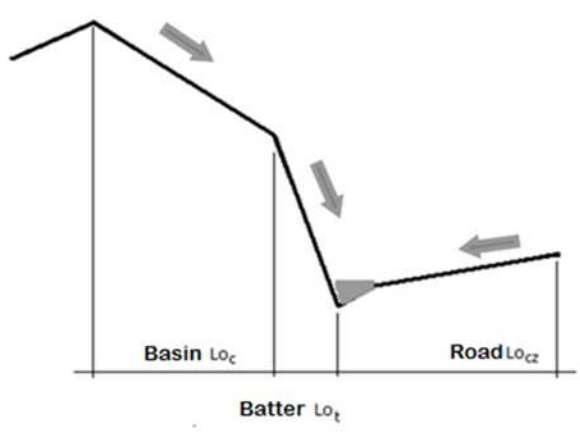


Fig. 5-2 Flow rates of contribution to the specific flow rate

The expression for calculating the specific flow rate is given by:

$$Q_e = Q_{Basin} + Q_{road} + Q_{backslope} \tag{5-2}$$

Where:
Q $_e$, is the specific flow rate.

To obtain these flow rates, the method considers a runoff coefficient K that varies from 0.05 to 0.90 depending on the type of surface. It is also considered a delay coefficient Cr oscillating in a range of 0007 to 0060, also according to the type of surface. The values of K and Cr for different surfaces are shown in Tables 5-3 and 5-4

Table 5-3 Runoff coefficients k (Chow, 2004)

SURFACE	K
Urban residential area: Only Residential houses Apartments with green areas Areas of commercial and industrial buildings	 0.30 0.50 0.90
Forested areas, depending on the soil	0.05 a 0.20
Parks, farmland and grassland	0.05 a 0.20
Asphalt or concrete pavement	0.85

Table 5-4 Delay coefficients Cr (Chow, 2004)

SURFACE	C_r
Flat asphalt surfaces	0.007
Concrete pavement	0.012
Gravel pavement	0.017
Very dense lawn	0.046
Grassland	0.060

The expressions used for the design of these works are:

$$t_c = \frac{527\mathrm{bL}_0^{\frac{1}{3}}}{[\mathrm{Ci}]^{\frac{2}{3}}} \tag{5-3}$$

Where:

$$b = \frac{0.0000276I + C_r}{S_0^{\frac{1}{3}}} \tag{5-4}$$

tc, time of concentration, in min.
i, rainfall intensity in mm / hour.
Lo, runoff length in meters.
C, runoff coefficient.
Cr, surface delay coefficient.

S0, average surface gradient.

Considering that i is obtained by a potential equation:

$$i = \frac{a}{(b + t_c)^c} \tag{5-5}$$

Where:
i, intensity corresponding to the duration t and return period Tr
a, b, c, coefficients obtained through least-squares analysis.
tc, time of concentration.

When interacting formulas Tc I and b the following equation is obtained:

$$t_c - 527\left(0.0000276\frac{a}{(t_c + b)^c} + Cr\right)\left(\frac{L_0}{S_0}\right)^{\frac{1}{3}}\left(\frac{1}{\left(\frac{ak}{(t_c + b)^c}\right)^{\frac{2}{3}}}\right) = 0 \tag{5-6}$$

With the above equation is found the value of t_c that meets the relationship, and if t_c is more than 5 minutes, the intensity obtained at that time is used, otherwise (less than 5 minutes t_c) the intensity obtained for 5 minutes is used.

With the intensity values obtained, as the case, we proceed to find the respective values of $Q_{Basin} + Q_{road} + Q_{slope}$ and the respective Q_e

To calculate the flow rate, the modified rational formula is used as follows:

$$Q = Ci\frac{L}{3600000} \tag{5-7}$$

Where:
Cr, runoff coefficient by area, road, batter and basin.
i, intensity mm / hr. also by area.
L, length, area in meters.

5.2.2. Cross Drainage

The cross drainage will be understood as a structure which aim to evacuate, leave or move surface flow from any natural or artificial traversing course or affecting the alignment and environment of a road project. In this regard, we can mention the culverts, cross tubes, box type sections, and domes.

As defined above there are several types of cross drains which are used in road projects, among the most commonly used are the gullies, culverts, domes and bridges.

For each of the above works there are particular for hydraulic analysis, then so will be developed those to take into account for structures of main roads.

5.2.2.1. Culverts

a) General Aspects

There are several criteria to define the limit for considering the construction of a culvert or dome or bridge, among these are:

- A culvert is any works that does not exceed a clear more than 6 m.
- The maximum flow rate to evacuate from the culvert must not exceed 15m^3/s.
- The maximum area of the cross section of the work must not exceed the equivalent area of a de182.88 (72”).

From the above criteria, the specialist in the area will define the limit of the works to be used depending to the particular conditions and the previous study of the site.

Other criteria to be taken into account in the design of the culvert are its location, alignment and gradient and protection works in the entrance and exit of these.

In general, the culverts are located in three places (Wright & Dizon, 2001)

- At the bottom of depressions where there are no natural watercourses.
- Where watercourses intersect roads
- In places where it is required for water to pass from the surface drainage led by gutters under roads and highways to adjacent properties.

Optimal placement for the culverts is achieved by following the alignment and the gradient of the natural riverbed. However, take note that the increase and decrease in the gradient influences the variation of the flow rate and it must be such as not excessively disturbs geomorphological processes such as erosion and sedimentation; therefore, gradient changes should be studied carefully, so as not to affect these processes that may lead to the collapse the structure.

b) Type and section

There are several types materials used in building culverts that can be used on road projects, within which are metal type, masonry, concrete and PVC culverts. In the case of this document, the concrete culverts were analyzed.

The most common sections are circular, rectangular and square.

On the need for cleaning and maintenance of the culverts, it is recommended to use a circular minimum cross section of 91.44 cm or 36”, or its area equivalent in other section. The minimum recommended value will also depend on the technical specifications or regulations that each country holds in.

It is important to install permanent culverts with a size large enough to dislodge the design floods plus the debris that can be anticipated.
At points where the construction of several parallel culverts (batteries tubes) is necessary, it is recommended to opt for a box type structure of a single course, because of the existing discontinuity in the clear, there are more chances of blockage at the entrance to the culverts.

c) Hydraulic design

evacuate or move a given flow rate, so it is important to have established and implemented the hydrologic calculations necessary for this purpose, along with all the considerations to be taken, as mentioned in the previous chapter of this manual, where one can find the most important considerations in this regard.

The hydraulic design of culverts includes the following general procedure:

- Obtain all the data of the site and draw the road cross section in the culvert place, including a profile of the canal watercourse.
- Set the elevations of headers of the culvert at the entrance and exit, and determine the length and gradient of the culvert.
- Determine the permissible water depth upstream and downstream probable for a design flood.
- Select the type and size for the culvert and the design features of the accessories that will fit the design flow under the conditions set.
- Consider the need for energy dissipaters and in the places where necessary, provide suitable protective devices to prevent erosion of the canal.

For the hydraulic analysis, two methods can be established, one is by means of the equation of Manning and the other is by controlling entry and exit. Some authors recommend that the analysis by the Manning equation to be used for obtaining a first approximation of the area of the necessary culvert, since it takes several simplifications (considering only uniform flow), but it is more advisable to use control methodologies of entry and exit.

Although it is simpler, the calculation methodology by Manning's equation than by the control methodology of entrance and exit by means of the Manning equation, we can get to overestimate the size of the culverts.

Between the two methods, it is always recommended to pay attention to the works of entrance and exit of the culvert but it is more critical if the control methodology for entrance and exit is used, as this methodology takes into account the conditions of these works.
With both methodologies, it is essential a maintenance plan but it becomes more important when designing this plan by control of entrance and exit.

Therefore it is important that the designer decides the most appropriate methodology depending on the conditions of the site and the cost benefit analysis carried out, but as mentioned above it is more advisable to carry out the design through the control of entrance and exit, as well as presented in the following item, these calculations are simplified by means of the nomograms developed to solve the equations of this methodology.

i. Design through Manning's equation[10]

An important aspect to consider for the hydraulic design using Manning's equation is to ensure that it works as an open riverbed. It is therefore necessary to establish a maximum height of the water level in the pipe, smaller than the diameter thereof, and it is assumed that the flow is uniform.

It is recommended that the maximum depth of the culvert does not pass on 2/3 of the diameter of the culvert, but the values can vary according to country specifications, which in the region varies between 0.8d and 0.9d, where d is the diameter of the culvert.

The Manning equation for the flow velocity and flow rate for a uniform system is provided by the following relationship:

$$v = \frac{1}{n} R_h^{\frac{2}{3}} S^{\frac{1}{2}} \tag{5-8}$$

Where:
v, the flow velocity, in m/s.
n, the Manning roughness coefficient, which recommended values are available from the table 5-5.
R_h, the hydraulic radius in m. (See calculation example for different shapes of canals on Table 5-8).
S, the gradient of the conduit, in m/m.

Once established the flow rate, by means of the continuity equation the conduit capacity is determined. This last one is expressed as follows:

$$Q = vA \tag{5-9}$$

Where:
Q, flow rate in m^3/seg.
A, the hydraulic area of the conduit in m^2.
v, is the flow velocity, a result of applying the Manning formula.

[10](Ministry of Transport and Communications, Perú, 2008)

Table 5-5 Manning n coefficients. Adapted from (Chow, 2004)

CANAL TYPE AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
Closed conduits flowing partially full			
Metal			
Steel Fluted Soldered Trimmed and spiral	 0.010 0.013	 0.012 0.016	 0.014 0.017
Molten metal Coated Uncoated	 0.010 0.011	 0.013 0.014	 0.014 0.016
Forged metal Black Galvanized	 0.012 0.013	 0.014 0.016	 0.015 0.017
Corrugated metal Sub drainage Rainwater drainage	 0.017 0.021	 0.019 0.024	 0.021 0.030
No metal			
Cement Polished surface Mortar	 0.010 0.011	 0.011 0.013	 0.013 0.015
Concrete Culvert line and free of debris Curved culvert, connections and some debris Well done Wastewater culverts	0.010 0.011 0.011 0.013	0.011 0.013 0.012 0.015	0.013 0.014 0.014 0.017
Other Manning coefficient values can be found at Open Canal Hydraulics ((Chow, 2004)) or other reference.			

Also, it is necessary to verify that the flow velocity is within certain limits because it can cause damages in the structure. Allowable velocity values according to the material of the culvert, shown in Table 5-6. The values shown in the table may vary according to the conditions of each country.

Table 5-6 Maximum admisible speed (m/s) in coated ducts (Ministerio de Transporte y Comunicaciones, Perú, 2008)

COATING TYPE	VELOCITY (m/s)
Concrete	3.0 – 6.0
Concrete brick	2.5 – 3.5
Stone masonry and concrete	2.0

It must also be verified that the minimum flow velocity within the conduit does not produce sedimentation that may affect a reduction in hydraulic capacity, recommending that the minimum velocity depends on the type of material from the culvert and to prevent sedimentation effects, it is recommended a minimum value of 0.5m/s, or the value recommended by type of project (duly justified) or the value given according to specifications of each country.

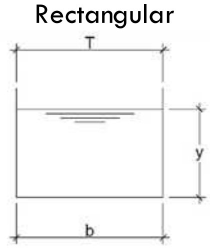
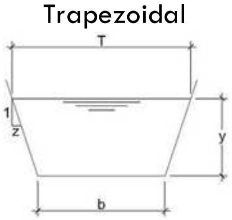
It also must take into account the flow velocity at the exit of the culvert, this velocity is generally greater than the one of runoff in the natural riverbed and should be limited in order to avoid undermining processes downstream the structure, and not affect its stability.

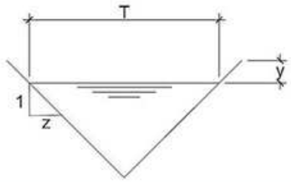
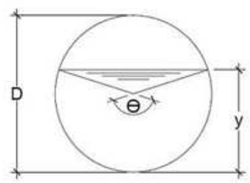
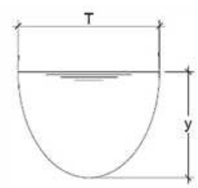
Below a table is presented with recommended maximum values of flow velocities according to the type of material where it moves.

Table 5-7 Maximum permissible velocities in unlined canals (Ministry of Transport and Communications, Perú, 2008)

TYPE OF TERRAIN	INTERMITTENT FLOW (m/s)	PERMANENT FLOW (m/s)
Fine sand (non-colloidal)	0.75	0.75
Sandy clay (non-colloidal)	0.75	0.75
Silty clay (non-colloidal)	0.9	0.9
Fine clay	1.0	1.0
Volcanic ash	1.2	1.0
Fine gravel	1.5	1.2
Hard clay (colloidal)	1.8	1.4
Graduate material (non-colloidal)		
From clay to gravel	2.0	1.5
From mud to gravel	2.1	1.7
Gravel	2.3	1.8
Coarse gravel	2.4	2.0
From gravel to stones (<15 cm)	2.7	2.1
From gravel to stones (> 20 cm)	3.0	2.4

Table 5-8 Formulas for obtaining different geometrical parameters of canal sections (Canals design small course)

SECTION	HYDRAULIC AREA (A)	WET PERIMETER (P_M)	HYDRAULIC RADIOUS (R_H)	WATER MIRROR (T)
Rectangular 	by	$b + 2y$	$\frac{by}{b + 2y}$	b
Trapezoidal 	$(b + zy)y$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$	$b + 2zy$

SECTION	HYDRAULIC AREA (A)	WET PERIMETER (P_M)	HYDRAULIC RADIOUS (R_H)	WATER MIRROR (T)
Triangular 	zy^2	$2y\sqrt{1+z^2}$	$\dfrac{zy}{2\sqrt{1+z^2}}$	$2zy$
Circular 	$\dfrac{(\theta-\sin\theta)D^2}{8}$	$\dfrac{\theta D}{2}$	$\left(1-\dfrac{\sin\theta}{\theta}\right)\dfrac{D}{4}$	$\left(\sin\dfrac{\theta}{2}\right)D$ ó $2\sqrt{y(D-y)}$
Parabólica 	$\dfrac{2}{3}Ty$	$T+\dfrac{8y^2}{3T}$	$\dfrac{2T^2y}{3T+8y^2}$	$\dfrac{3A}{2y}$

ii. Control of entrance and exit design

The type of flow, which occurs in a culvert, depends on the total amount of energy available between the entrance and the exit. The available energy is mainly composed of the potential energy, which is the difference between the head and discharge (usually under pond conditions, the rate at the entrance is small and the velocity or kinetic energy can be assumed equal to zero). All available energy is spent entirely on the flow that occurs naturally. Thus, the energy is dissipated upon entering, by friction, with charging velocity and with depth.

The flow characteristics and capacity of a culvert are determined by the location of the *control section*. It can be said that the control section is the part of the culvert that operates at maximum flow; the other parts of the system have a larger capacity than they actually used.

Laboratory tests and field studies indicate that roads culverts operate with two types of control: *entrance and exit*. (Wright & Dizon, 2001)

The design procedure presented here is developed by the FHWA and published in the Hydraulic Design of Highway Culverts. The control section the culvert is used to classify the different flows in it. The location for which there is a unique relationship between the expense or flow rate and the depth of upstream flow is the control section. When the flow is determined by the geometry of the entrance, then the control section is the entrance to the culvert, that is, the upstream end of the culvert and the flow *is controlled at the entrance*. When the flow is governed by a combination of the head of water at the site of download, the *entrance of the culvert and the characteristics of the culvert cylinder, the flow is controlled* at the exit. Several design letters and nomograms are used in the design process, developed from a combination of theory and many hydraulic test results; these procedures are shown below.

- **Entrance control. (Garber & Hoel, 2007)**

The culverts flow operating under control of entrance conditions, it is supercritical with high velocities and low depths. Fig. 5-3 shows four different flows under entrance control. The flow type depends on whether the entrance, the exit or both are submerged in the culvert. In Fig. 5-3 (a), both the entrance and the exit are above the water surface. In this case, the flow within the culvert is supercritical; the culvert is partially filled throughout its length and the depth of flow approaches normally in the exit end. In Fig. 5-3 (b), only the downstream end (exit) of the culvert is submerged, but this does not produce an exit control. The flow in the culvert shortly after the entry of the same is supercritical, and a hydraulic jump occurs within the culvert. In Fig. 5-3 (c) there is the entry end of the culvert submerged, with water free flowing in the exit. The culvert is partially filled throughout its length and the flow is supercritical within the same, as the critical depth is located just after the culvert entrance. The depth of the flow at the exit of the culvert also approaches normally. Fig. 5-3 (d) shows immersed both the entrance and the exit of the culvert, but the culvert is partially filled in a part of its length. A hydraulic jump within the culvert occurs, causing the culvert to fill throughout the remaining length. Under these conditions, pressures below atmospheric may develop, so that an unstable situation is created with the culvert ranging from partially full flow and full flow. This can be avoided by installing an intermediate entrance.

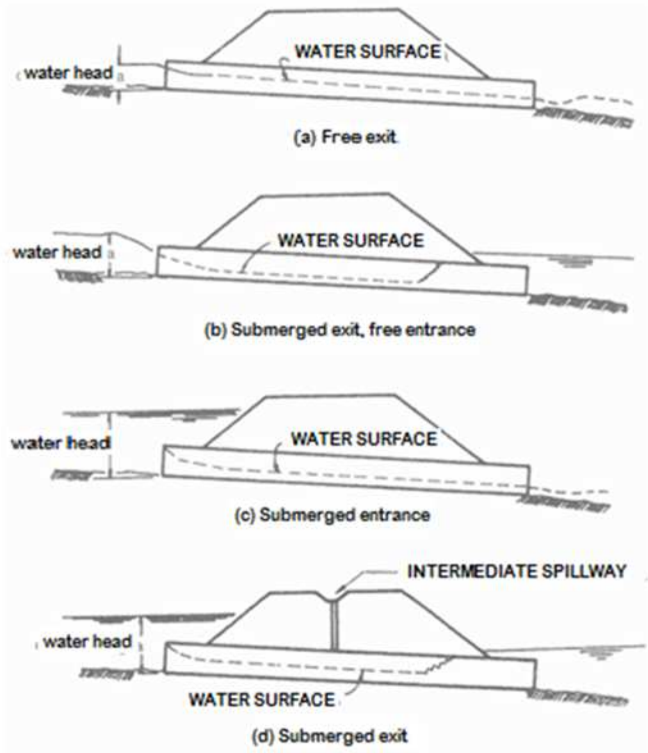


Fig. 5-3 Types of entrance control (Garber & Hoel, 2007)

Several model tests have been made to determine the relationship between the head of water (water depth above the entrance point of the culvert) and flow to culverts in entrance control condition. The basic condition is used if the entrance is immersed or not.

For the non-submerged condition, two conditions have been developed; the equation 5.10 is based on the specific load for the critical depth, and the second 5.11 is an exponential equation similar to a landfill. The first equation has more theoretical support, but the second is easier to use. Equation 5.12 is for a submerged condition.

$$\frac{HW_i}{\mathrm{D}} = \frac{H_i}{D} + K\left[\frac{\mathrm{Q}}{A.D^{0.5}}\right]^M - 0.5\mathrm{S} \qquad (5\text{-}10)$$

$$\frac{HW_i}{\mathrm{D}} = K\left[\frac{\mathrm{Q}}{A.D^{0.5}}\right]^M \qquad (5\text{-}11)$$

$$\frac{HW_i}{\mathrm{D}} = c\left[\frac{\mathrm{Q}}{A.D^{0.5}}\right]^2 + \mathrm{Y} - 0.5\mathrm{S} \qquad (5\text{-}12)$$

Where:
Hw_i, depth of load or head of water needed above the entrance control section, in feet [11]
D, internal height of the cylinder of the culvert, in feet.
V, flow velocity (feet/seconds).
V_c, critical velocity (feet/seconds).

[11] 1 foot = 1/3.2808 m

g, 32.2 feet/seconds^2.

H_c, specific head of the critical depth, ie, $H_c = d_c + \left(\frac{V_c^2}{2g}\right)$.

d_c, critical depth (feet).
A, total area of the cross section of the culvert cylinder (feet^2).
S, cylinder gradient of the culvert (feet/foot).
K,M,c,Y, constants shown in Table.5-9.

Note that the last term (-0.5S) in equations 5.10 and 5.12, must be replaced by +0.7S when chamfered corners are used. Equations 5.10 and 5.11 are applicable to about $Q/A.D^{0.5}=3.5$. Equation 5.12 is applicable above approximately $Q/A.D^{0.5}= 4$.

Table 5-9 Coefficients for the design equations of entrance control (Garber & Hoel, 2007)

SHAPE & MATERIAL	DESCRIPTION OF THE ENTRANCE EDGE	SHAPE	NON SUBMERGED		SUBMERGED	
			K	M	C	Y
Circular	Wall with squared edge	1	0.0098	2.0	0.0398	0.67
Concrete	Gouging end with head wall Outlet with slotted end	1	0.0078 0.0045	2.0 2.0	0.292 0.0317	0.74 0.69
Circular	Head	1	0.0078	2.0	0.0379	0.69
CMP	With chamfered joint until providing the gradient Outlet	1	0.0210 0.0340	1.33 1.50	0.0463 0.0553	0.75 0.54
Circular	Beveled rings, bezels of 45° Beveled rings, bezels of 33.7°	1	0.0018 0.0018	2.50 2.50	0.0300 0.0243	0.74 0.83
Rectangular	Walls with splayed eaves, angles of 30° and 75°	1	0.026	1.0	0.0385	0.81
Box	Walls with splayed eaves, angles of 90° and 15° Walls with splayed eaves to 0°	1	0.061 0.061	0.75 0.75	0.0400 0.0423	0.80 0.82
Rectangular	Walls with splayed eaves to 45° d=0.0430	2	0.510	0.667	0.0309	0.80
Box	Walls with splayed eaves to 18° to 33.7° d=0.0830	2	0.486	0.667	0.0249	0.83
Rectangular	Head Wall to 90° with chamfers to $^3/_4$"	2	0.515	0.667	0.0375	0.79
Box	Head Wall to 90° with bezels to 45° Head Wall to 90° with bezels to 33.7°	2	0.495 0.486	0.667 0.667	0.0314 0.0252	0.82 0.865
Rectangular	Chamfers to $^3/_4$"; head with skew to 45°	2	0.522	0.667	0.0402	0.73
Box	Chamfers to $^3/_4$"; head with skew to 30° Chamfers to $^3/_4$"; head with skew to 15° Bezels to 45°; head with skew from 10° to 45°		0.533 0.545 0.498	0.667 0.667 0.667	0.0425 0.04505 0.0327	0.705 0.68 0.75

SHAPE & MATERIAL	DESCRIPTION OF THE ENTRANCE EDGE	SHAPE	NON SUBMERGED		SUBMERGED	
			K	M	C	Y
Rectangular	Walls with splayed eaves, 45° no transition	2	0.497	0.667	0.0339	0.803
Box	Walls with splayed eaves, 18.4° no transition	2	0.493	0.667	0.0361	0.806
Chamfers to $^{3}/_{4}$"	Walls with splayed eaves, 18.4° no transition Skewed cylinder of 30°		0.495	0.667	0.0386	0.71
Rectangular	Walls with splayed eaves 45° with transition	2	0.497	0.667	0.0302	0.835
Box	Walls with splayed eaves 33.7° with transition		0.495	0.667	0.0252	0.881
Superior Bezels	Walls with splayed eaves 18.4° with transition		0.493	0.667	0.0227	0.887
CM Boxes	Head to 90° Outlet of thick wall Outlet of thin wall	1	0.0083 0.0145 0.0340	2.0 1.75 1.5	0.0379 0.0419 0.0496	0.69 0.64 0.57
Horizontal	Head with squared edge	1	0.0100	2.0	0.0398	0.67
Ellipse	Head / wall with slotted end		0.0018	2.5	0.0292	0.74
Concrete	Outlet with slotted end		0.0045	2.0	0.0317	0.69
Vertical	Head / wall with squared edge	1	0.0100	2.0	0.0398	0.67
Ellipse	Head / wall with slotted end		0.0018	2.5	0.0292	0.74
Concrete	Outlet of with slotted end		0.0095	2.0	0.0317	0.69
Bend pipe	Outlet of with slotted end, head of 90°	1	0.0083	2.0	0.0379	0.69
Corner to 18°	Chamfered up to the gradient		0.0300	1.0	0.0463	0.75
CM Radius	Outlet		0.0340	1.5	0.0496	0.57
Bend pipe	Outlet	1	0.0296	1.5	0.0487	0.55
Corner to 18°	Non-beveled		0.0087	2.0	0.0361	0.66
CM Radius	Beveled to 33.7°		0.0030	2.0	0.0264	0.75
Bend pipe	Outlet	1	0.0296	1.5	0.0487	0.55
Corner to 31°	Non-beveled		0.0087	2.0	0.0361	0.66
CM Radius	Beveled to 33.7°		0.0030	2.0	0.0264	0.75
CM Arc	Head to 90° Chamfered up to the gradient Outlet of thin wall	1	0.0083 0.0300 0.0340	2.0 1.0 1.5	0.0379 0.0463 0.0496	0.69 0.75 0.57
Circular	Entrance gullet smoothly splayed Entrance gullet abruptly splayed	2	0.534 0.519	0.555 0.64	0.0196 0.0289	0.89 0.90
Elliptical	Splayed entrance with beveled edges	2	0.536	0.622	0.368	0.83
Entrance face	Splayed entrance with squared edges Splayed entrance with thin edge outlet		0.5035 0.547	0.719 0.80	0.0478 0.0598	0.80 0.75

SHAPE & MATERIAL	DESCRIPTION OF THE ENTRANCE EDGE	SHAPE	NON SUBMERGED		SUBMERGED	
			K	M	C	Y
Rectangular	Splayed entrance gullet	2	0.475	0.667	0.0179	0.97
Rectangular	Splayed sides with less favorable edges	2	0.56	0.667	0.0466	0.85
Concrete	Splayed sides with more favorable edges		0.56	0.667	0.0378	0.87
Rectangular	Less favorable edges – splayed gradient	2	0.50	0.667	0.0466	0.65
Concrete	More favorable edges – splayed gradient		0.5	0.667	0.0378	0.71

Several nomograms have been developed for different shapes of culvert based on these equations. Fig. 5-4 presents the nomograph for culverts with a rectangular box for the entrance control, with walls with splayed eaves and with beveled edge at the top of the entrance, and Fig. 5.5 shows nomograph for a circular tube culvert under entrance control (with n=0.012), These nomograms are in English system but the source used for further reference for other sections and nomographs in international system you can consult the *Hydraulics Design of Highway Culverts* from FHWA.

These figures are used to determine the depth of the head of water required to accommodate the design flow through the selected configuration of the culvert under entrance control conditions.

It is important to note that for a good design, one should establish limits on the ratio of the entrance hydraulic load and the diameter (Hw/D) which as a reference on this value may be around 1.2< Hw/D <1.5 (according to AASHTO), or in other cases the height of the upper bound of the entrance head of the culvert is taken, or this relationship is established leaving a safety distance to the grade line of the pavement structure of the road; these factors are set according to the regulations of each country and considerations from the designer.

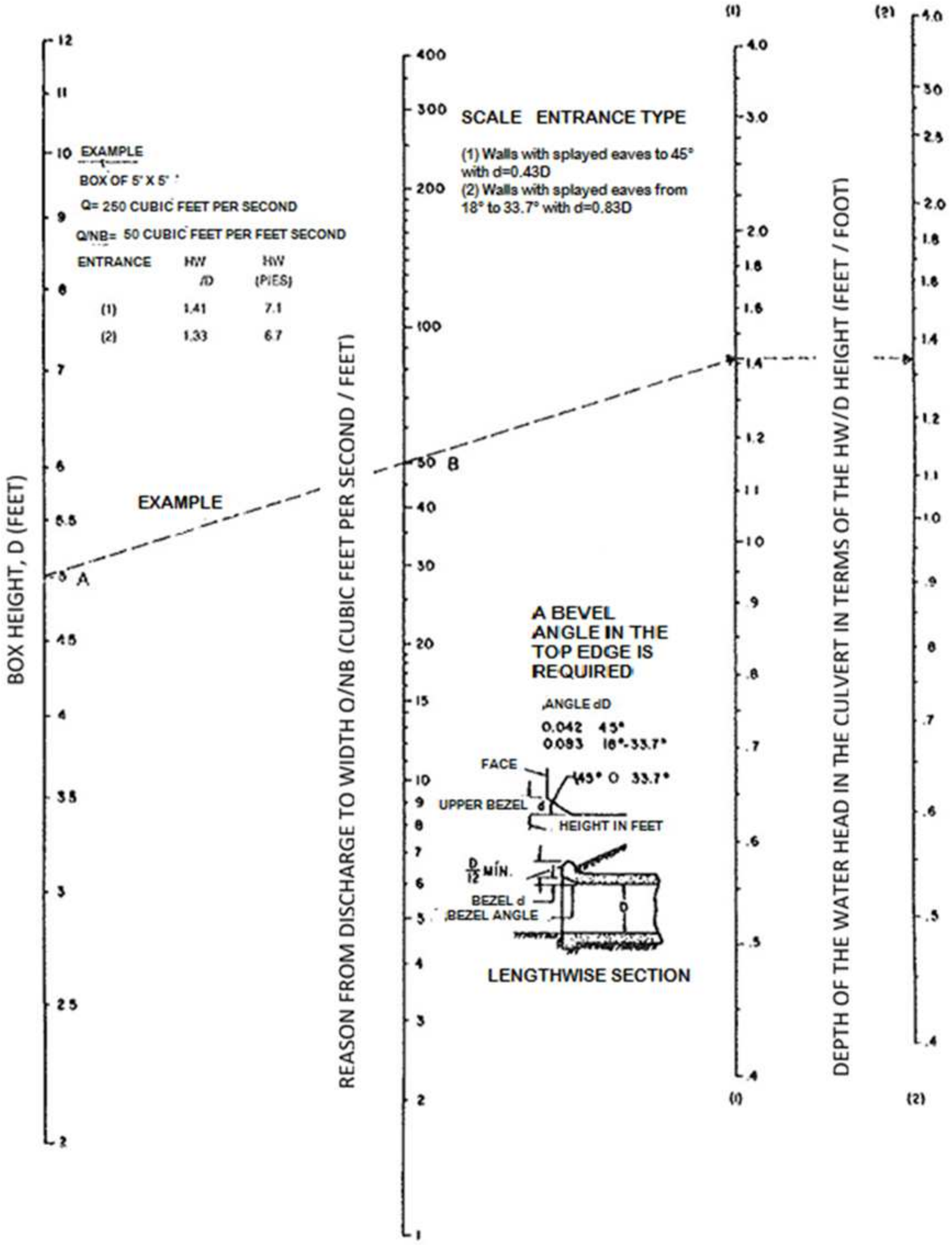


Fig. 5-4 Depth of water head for the entrance control, of rectangular box culverts, walls with splayed eaves, from 18° to 33.7° and 45° with beveled edge at the top of the entrance (for further reference, consult Hydraulics Design of Highway Culverts from FHWA).

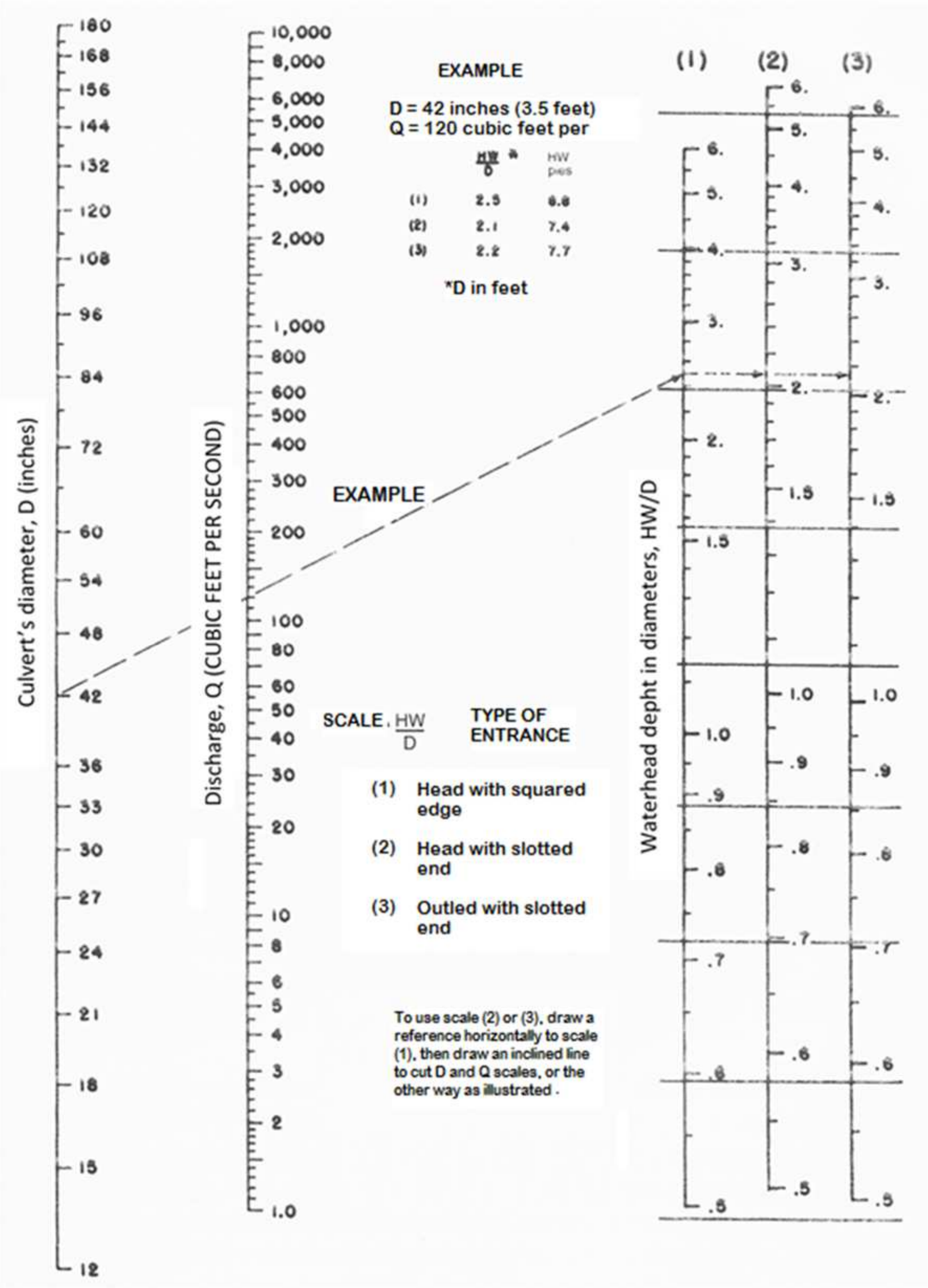


Fig. 5-5 Headwater depth for culverts of concrete pipes with entrance control (for further reference, see Hydraulics Design of Highway Culverts from FHWA).

- **Exit control design (Garber & Hoel, 2007)**

A culvert flows under exit control when the cylinder is not capable to transport as much flow as possible to the opening entrance can receive. In Fig. 5-6 shows different types of low flow conditions of entrance control where the control section is located at the downstream end of the culvert or beyond. In Fig. 5-6 (a), both the entrance and the exit are immersed, and water flows under pressure throughout the entire length of the culvert, completely filled. In Fig. 5-6 (b) shows the non-submerged entrance and the submerged exit. In Fig. 5-6 (c), the exit is not submerged and the culvert flows filled along its entire length, due to the height of the water head. In Fig. 5-6 (d), the entrance to the culvert is submerged and the exit is not submerged, and the depth of water at the exit is low. Therefore, the culvert flows partially filled. Flow is also subcritical throughout the length part of the culvert, but the critical depth is just upstream of the exit. In Fig. 5-6 (e) both non-submerged entrance and exit with the partially filled culvert throughout its entire length, and subcritical flow.

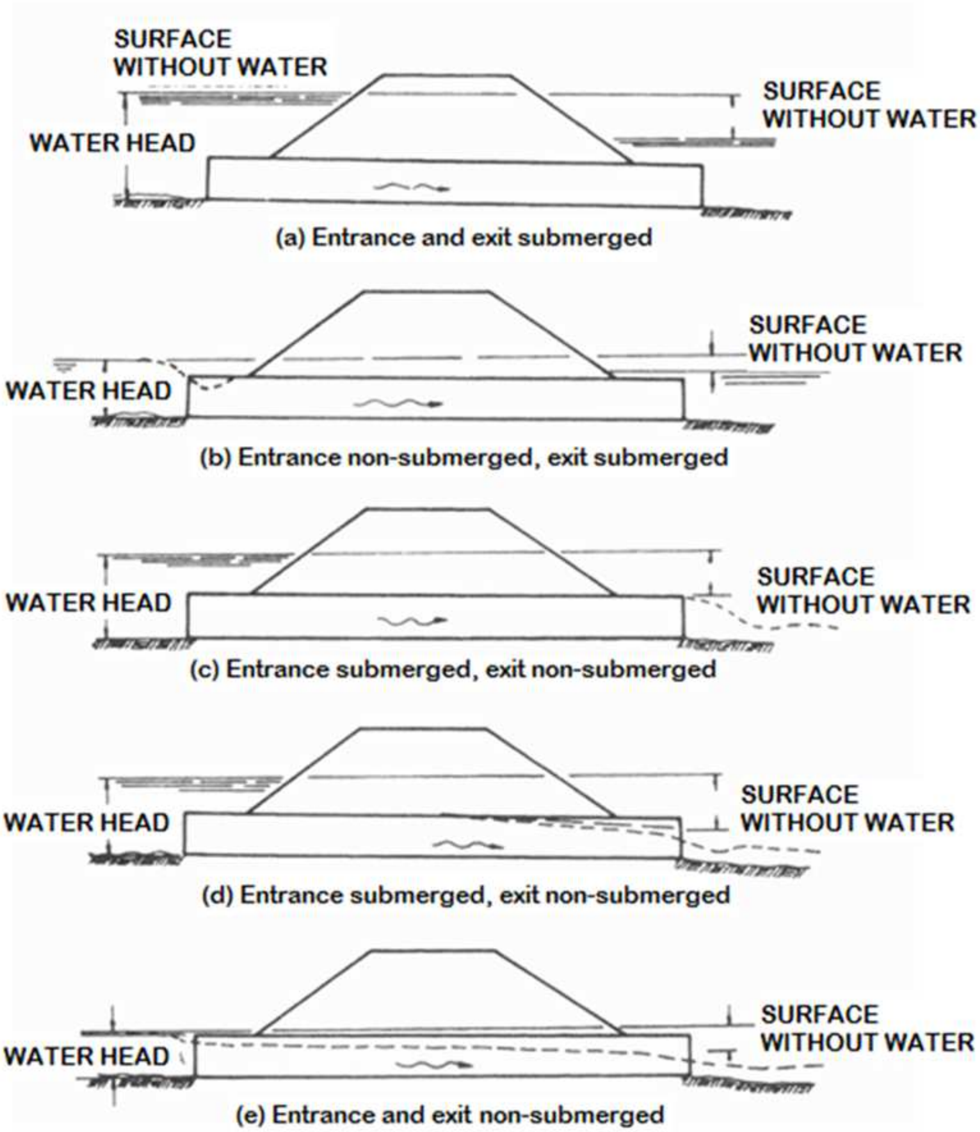


Fig. 5-6 Types of exit control (Garber & Hoel, 2007).

The hydraulic analysis of the culverts flowing under exit control is based on energy balance. The total loss of energy through the culvert is given as:

$$H_L = H_e + H_f + H_o + H_b + H_j + H_g \qquad (5\text{-}13)$$

Where:
H_L, total required energy.
H_e, loss of energy at the entrance
H_f, loss by friction.
H_o, loss of energy at the exit.
H_b, loss by change of course.
H_j, loss of energy at the joint
H_g, loss of energy at the security grids

Without taking into account the losses due to change of course, joint and grids, the total discharge loss is given as:

$$H_L = \left(1 + k_e + \frac{29n^2L}{R_h^{1.33}}\right)\frac{V}{2g}^2 \qquad (5\text{-}14)$$

Where:
k_e, factor based on different entrance configurations (Table 5-10).
n, Manning coefficient for culverts (Table 5-11).
R_h, hydraulic radius of the complete cylinder of the culvert.
L, cylinder length of the culvert.
V, cylinder velocity.

Table 5-10 Coefficient of losses in entrance. (Norman & Johnston, 1985)

TYPE OF STRUCTURE AND DESIGN OF THE INLET	COEFFICIENT k_e
Pipe, concrete	
Outlet of landfill, wedged end (slotted end)	0.2
Outlet of landfill, squared cut end	0.5
Head alone or head and eave walls Pipe with embedded end (slotted end) Squared edge Rounded (radio = $1/_2\,D$)	 0.2 0.5 0.2
Chamfered to match the gradient batter	0.7
End section that is equal with the batter of the landfill	0.5
Beveled edges, bezels of 33.7° or 45°	0.2
Entrance splayed at the edges or gradient	0.2
Pipe or vended pipe, corrugated metal	
Outlet of the landfill (without head)	0.9
Head or head and walls eaves with square edges	0.5
Chamfered to match the batter of the landfill, coated or uncoated batter	0.7
End section to match the batter of the landfill	0.5
Beveled edges, bezels of de 33.7° or 45°	0.2
Entrance splayed at the edges or gradient	0.2

TYPE OF STRUCTURE AND DESIGN OF THE INLET	COEFFICIENT k_e
In box, reinforced concrete	
Head parallel to the landfill (without walls with eaves,) Edge squarely in 3 banks Rounded in 3 edges with a radius of $^1/_{12}$ of the dimension of the cylinder, or beveled edges in 3 banks	0.5 0.2
Walls with eaves from 30° to 75° in regard of the cylinder Squared edge in the crown Rounded crown edge with a radius of $^1/_{12}$ of the dimension of the cylinder, or with the upper beveled edge	0.4 0.2
Walls with eaves from 10° to 25° in regard of the cylinder Squared edge in the crown	0.5
Walls with parallel eaves (extending sides) Squared edge in the crown	0.7
Entrance splayed at the edges or gradient	0.2

Table 5-11 Manning coefficients for culverts. (Norman & Johnston, 1985)

TYPE OF CONDUIT	WALL AND JOINT DESCRIPTION	MANNING N
Concrete pipe	Good joints, smooth walls Good joints, rough walls Deficient joints, rough walls	0.011-0.013 0.014-0.016 0.016-0.017
Concrete box	Good joints walls with smooth finish Deficient joints, rough walls without finish	0.012-0.015 0.014-0.018
Pipes and boxes of corrugated metal, corrugated ring (the Manning's n varies with the size of the cylinder)	Corrugated of 2 $^2/_3$ by $^1/_2$ inches Corrugated of 6 by 1 inch Corrugated of 5 by 1 inch Corrugated of 3 by 1 inch Corrugated of structural plate of 6 by 2 inches Corrugated of structural plate of 9 by 1 inch 2 $^1/_2$ inches	0.027-0.022 0.025-0.022 0.026-0.025 0.028-0.027 0.035-0.033 0.037-0.033
Pipes made of corrugated metal, corrugated helical, from circular flow to filled tube	Corrugated of 2 $^2/_3$ by $^1/_2$ inches wide of a 24-inch plate	0.012-0.024
Spiral corrugated metal pipe	Slot of $^3/_4$ by $^3/_4$ inches with a spacing of 12 inches, good joints	0.012-0.013

Fig. 5-7 is a scheme of energy lines of the gradient of a culvert the flowing filled. If the total of energies at the entrance and exit are equal, we have:

$$HW_0 + \frac{v_u^2}{2g} = TW + \frac{v_d^2}{2g} + H_L \tag{5-15}$$

Where:
HW_0, water head depth above the exit point (feet).
V_u, approach velocity.
TW, water depth at the site of discharge above the exit (feet).
V_d, downstream velocity (feet/seconds).
H_L, sum of all losses.

When the velocity of the load is discarded at the entrance and downstream, is obtained:

$$HW_0 = TW + H_L \quad (5\text{-}16)$$

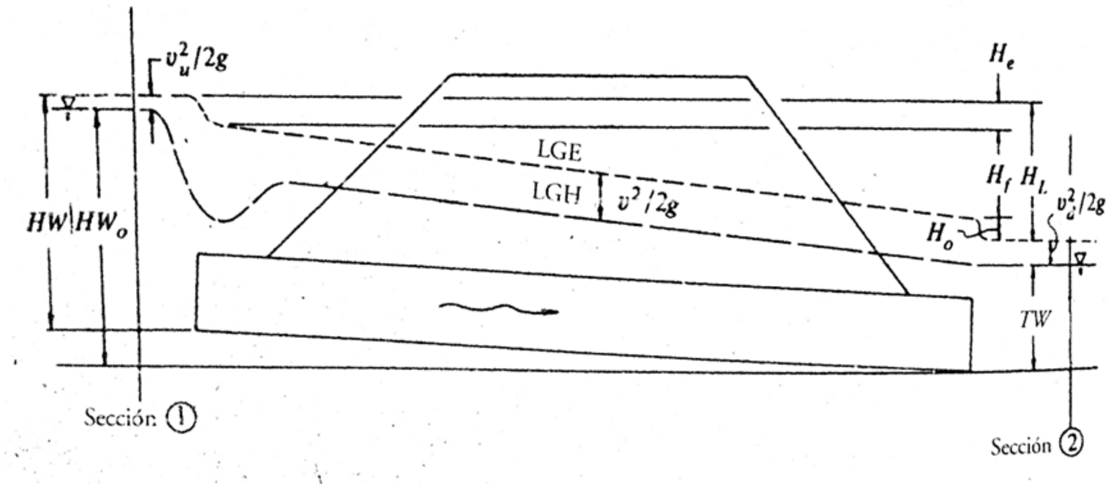


Fig. 5-7 Gradient energy line (LGE) and hydraulic grade line (LGH) for filled tube flow. (J.M. Norman, R. J. Houghtalen, W.J. Johnston. Hydraulic design of culverts for roads)

Note that the equations 5.13, 5.14, 5.15 and 5.16 are developed for culverts with filled flow and therefore are applicable to the conditions shown in Figures Fig. 5-6 (a), Fig. 5-6 (b), Fig. 5-6 (c) in other cases may require additional calculations of greater complexity.

Nomographs have also been developed to solve the equation 5.15, for different settings for exit control culverts, in the nomographs are only considered losses the ones by entrance, by friction and by exit. In Figures 5-8 and 5-9 are shown as an example, nomographs for concrete box culvert and a circular concrete culvert (with n = 0.012) with a sourced used in English system, but for further reference for other sections and nomographs in the international system, you can consult the *Hydraulics Design of Highway Culverts* from FHWA.

Also in Figures 5-10 and 5.11 there are figures for the critical depth for the critical depth for these types of culverts, since it must meet the critical depth, it must not exceed the diameter of the culvert.

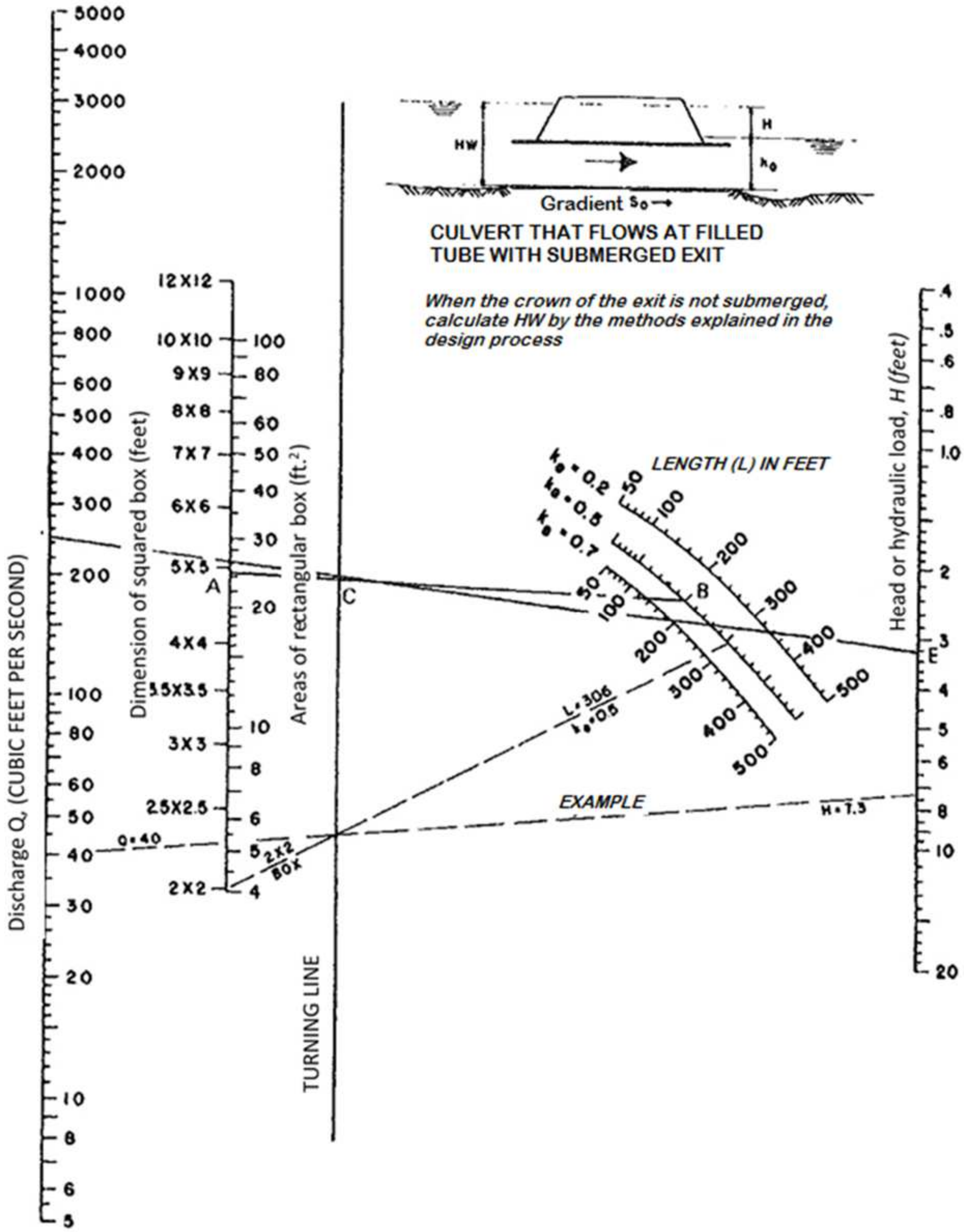


Fig. 5-8 Headwater depth of water for concrete box culverts flowing at filled tube. For further information, consult the Hydraulics Design of Highway Culverts from FHWA (J.M. Norman, R. J. Houghtalen, W.J. Johnston Hydraulics Design of Highway Culverts)

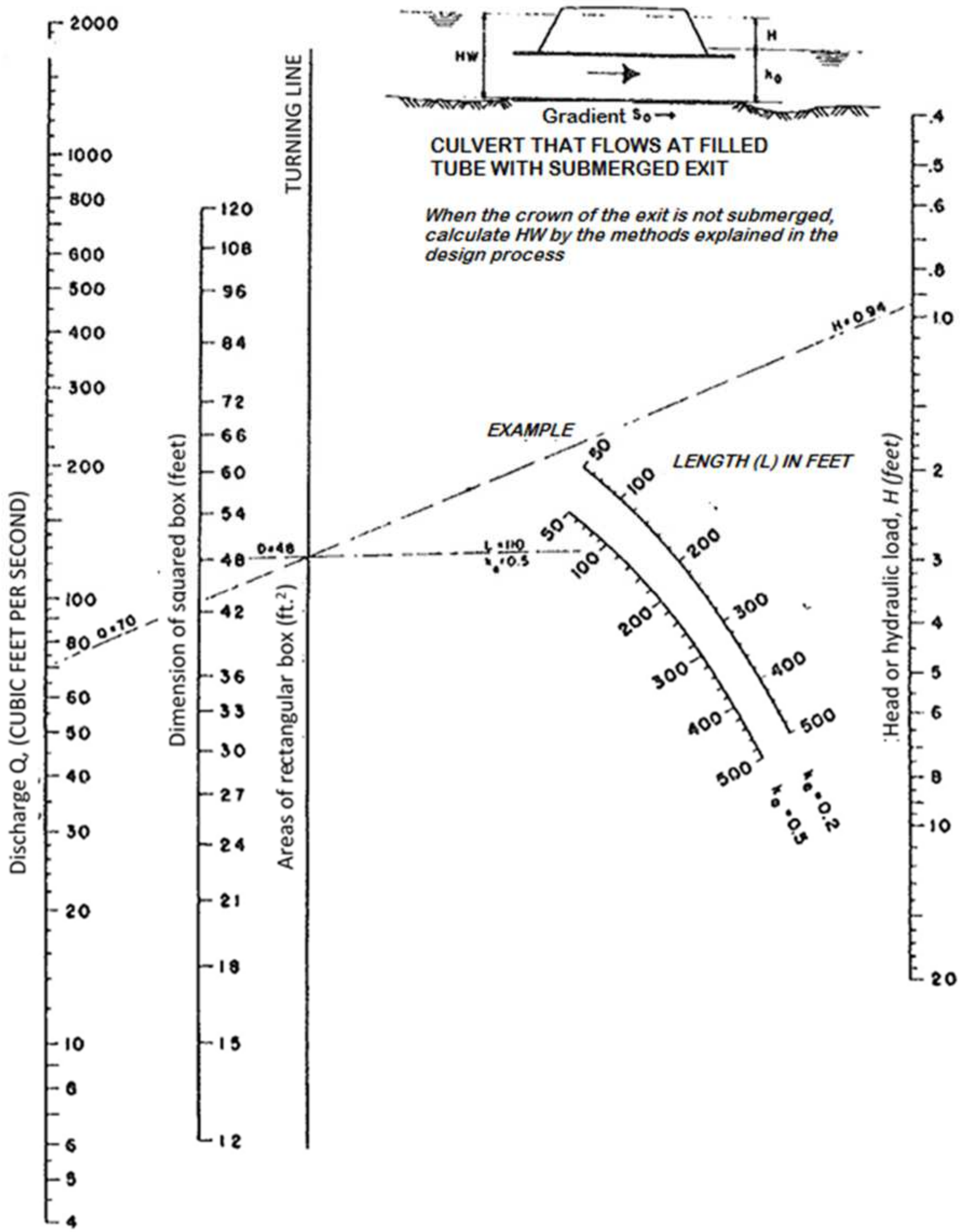


Fig. 5-9 Headwater depth for concrete pipes flowing at a filled tube, in English system. For further information, consult Hydraulics Design of Highway Culverts from FHWA. (J.M. Norman, R. J. Houghtalen, W.J. Johnston. Hydraulics Design of Highway Culverts)

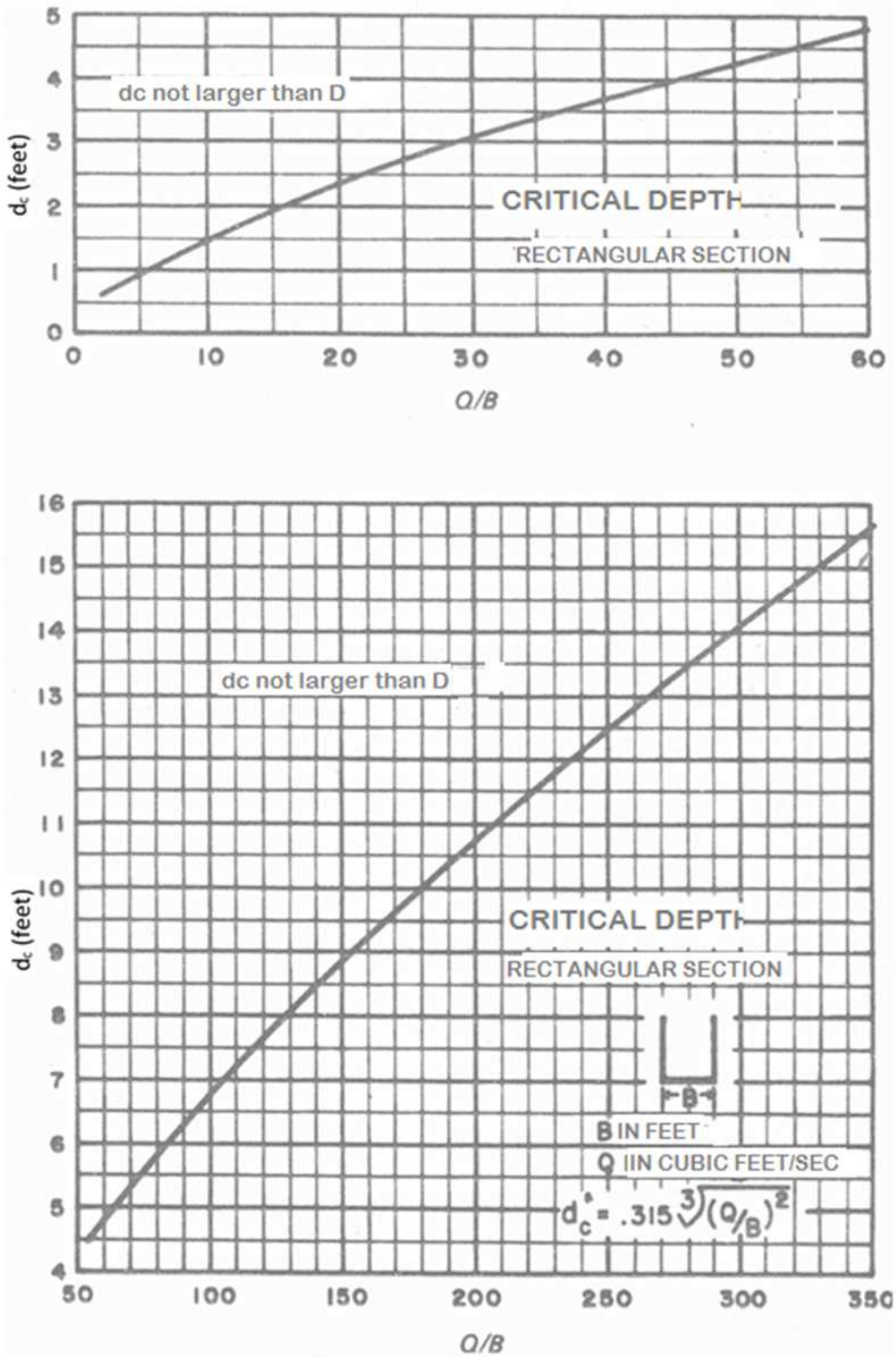


Fig. 5-10 Critical depth for rectangular sections. (J.M. Norman, R. J. Houghtalen, W.J. Johnston. Hydraulics Design of Highway Culverts)

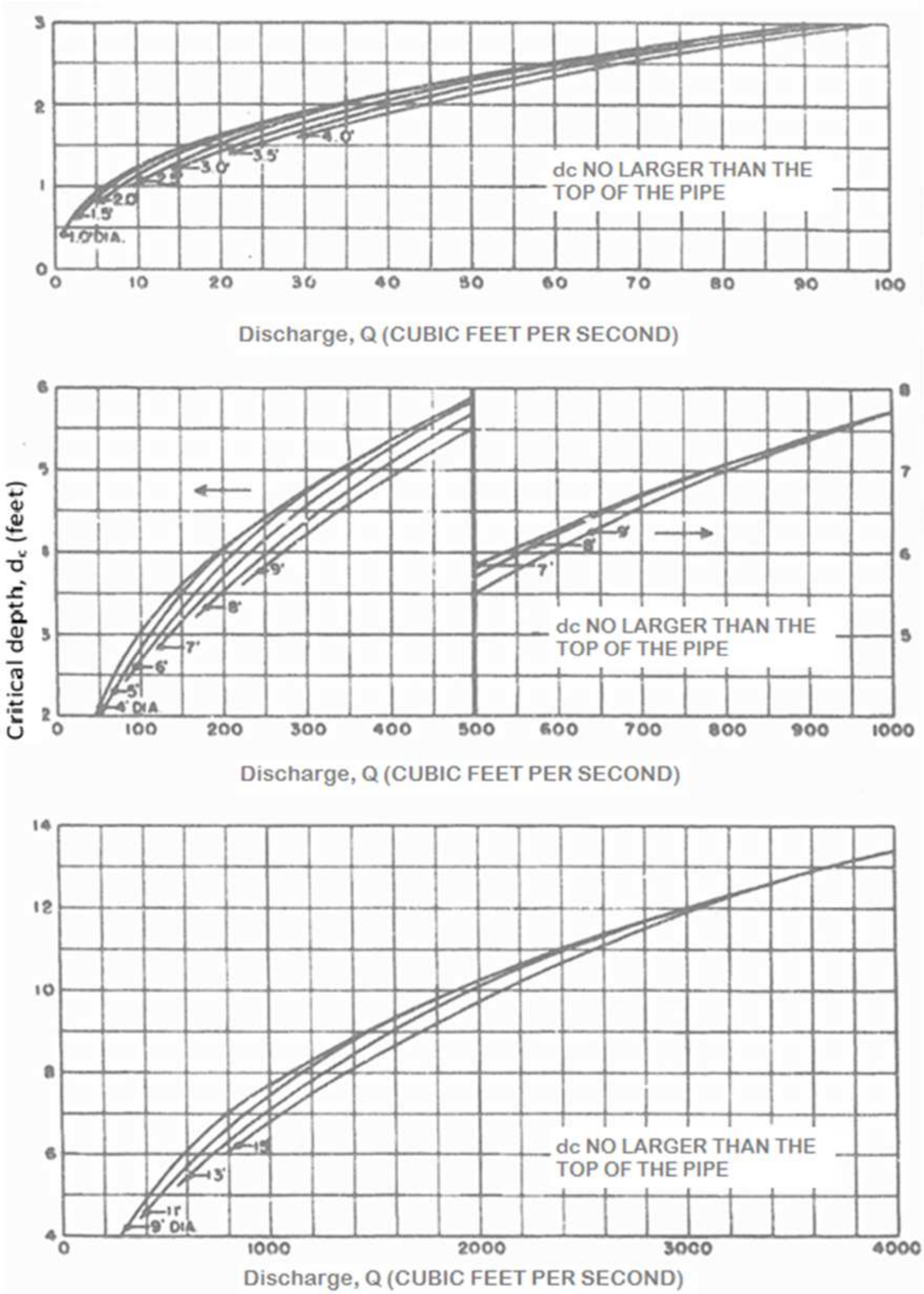


Fig. 5-11 Critical depth for circular pipes. (J.M. Norman, R. J. Houghtalen, W.J. Johnston. Hydraulics Design of Highway Culverts)

In a general way can be established the following procedure for hydraulic analysis of culverts.

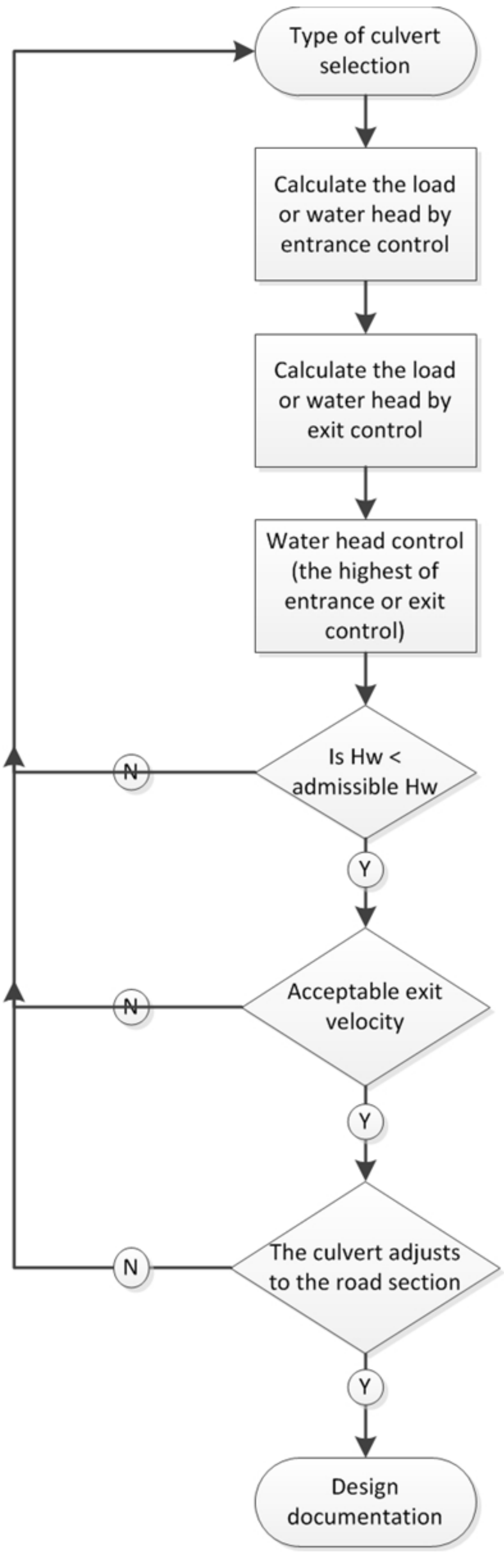


Fig. 5-12 Flowchart for the culvert design (developed from (FHWA, 2012).

5.2.2.2. Bridges

With respect to these types of works, because of its complexity and for being the kind of works that individually represent a good percentage of the investment to be made in a road project, the analysis of these should be performed as part of a specific study for each structure, and also applying the use of specialized software for its hydrologic and hydraulic analysis, so here are presented some considerations to take into account during the study of these along with a brief explanation of the hydraulic structures.

Depending on the stage of the design of the bridge, some recommendations must be taken for the proper performance of this so here are some guidelines or recommendations thereon (DACGER-MOPTVDU, 2014); it is also recommended to refer the considerations made in the Central American Manual of Risk Management in Bridges, developed by the Central American Economic Integration System (SICA)

a) Guidelines for superstructure

The superstructure of the bridge should be placed, whenever possible, at an elevation higher than the approach areas of the road (see Fig. 5-12), which allows that during an extreme event, the water exceeds the access landfills so that the hydraulic forces on the bridge are alleviated. This is particularly important in streams that carry large amounts of waste and that block the passage of water beneath the superstructure.

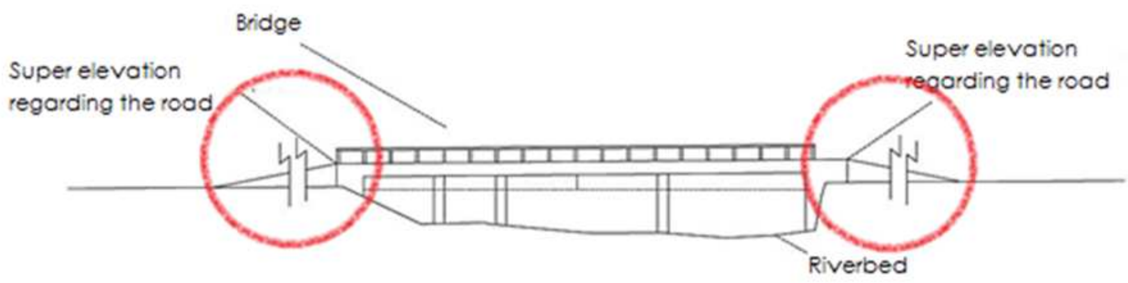


Fig. 5-13 Bridge scheme on super elevation with respect on the approach ends.

Due to the risk taken during a flood higher than the design, it is recommended that the superstructure is fastened to the substructure with some type of anchoring system to protect the structure in case some drag, thrust, impact or forces by flotation are presented, as a result of an increase in the level of the river, exceeding the grade level of the bridge.

b) Guidelines for substructure

For the foundation design of the intermediate piers and abutments or bastions, consider the use of deep foundations (piles), especially when the soil does not have the mechanical properties to support or sustain a shallow foundation, and is susceptible to erosion and scour (see Fig. 5-14). If the foundation bed is shallow rock, then a shallow foundation will be the most adequate.

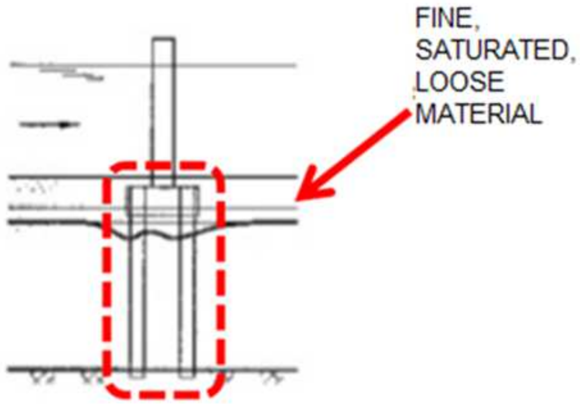
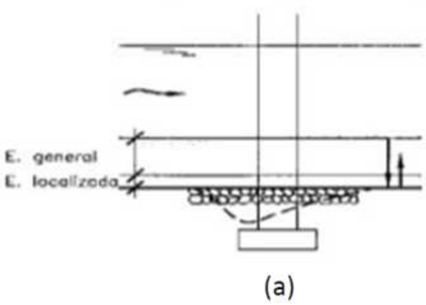


Fig. 5-14 Deep foundation use. (Martin Vide, 2003)

Consider in the design of elements of the substructure of a bridge (abutments or bastions, piles, and their respective foundations) the effect of drag loads, thrust and impact.

For intermediate supports on existing structures, it is recommended to contemplate the necessary protection depending on the type of flow that has grown during peak and normal courses. To do this, consider the use of protective devices, such as placing a rock fill (or pier) about the piles (see Fig. 5.15a). Note that such devices should be placed where they are effective, that is, to a depth given by undermining projected levels (general and shrink erosion). (See Fig. 5.15b).



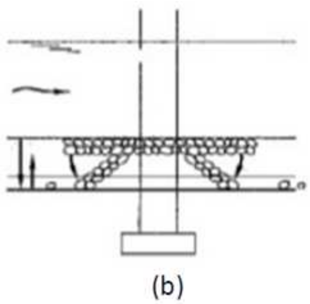


Fig. 5-15 Rock fill protection scheme: (a) Example of rock fill use; (b) Incorrect rock fill location. (Martin Vide, 2003)

Regardless of the alignment of the bridge regarding the course of flow of the river to cross, the position and orientation of the intermediate supports of the bridge should be parallel to the course of flow of the river (see Fig. 16), also considering the construction of these elements with hydrodynamic shapes, in order to reduce any effect of scour that the flow may produce on these elements in their bases. (On this issue, we recommend consulting the document "Scour at Bridges" by Dr. Maria Guevara from the University of Cauca 2001).

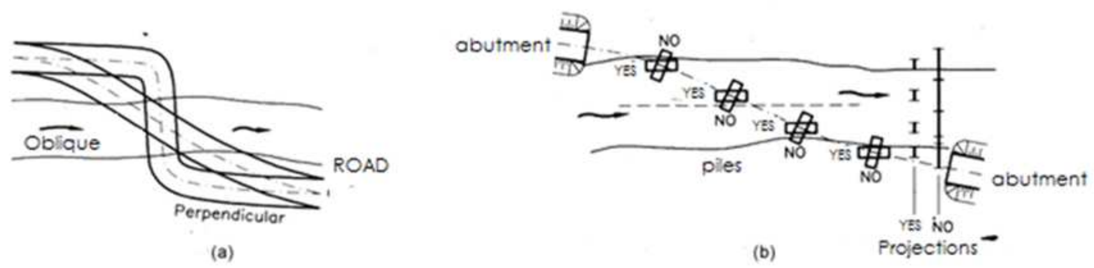


Fig. 5-16 Alignment of intermediate supports of bridges: (a) Alignment of the bridge with respect to the river; (B) Alignment of intermediate supports. (Martin Vide, 2003)

Bridge abutments must be located at least on the edge of riverbed width and according to the following (See Fig. 5-17):

- The location of the front of the abutment wall must be at least at the intersection of the riverbank with high water level (N.A.M.).
- The foot of the batter of the road landfill, if any, shall not project into the river.

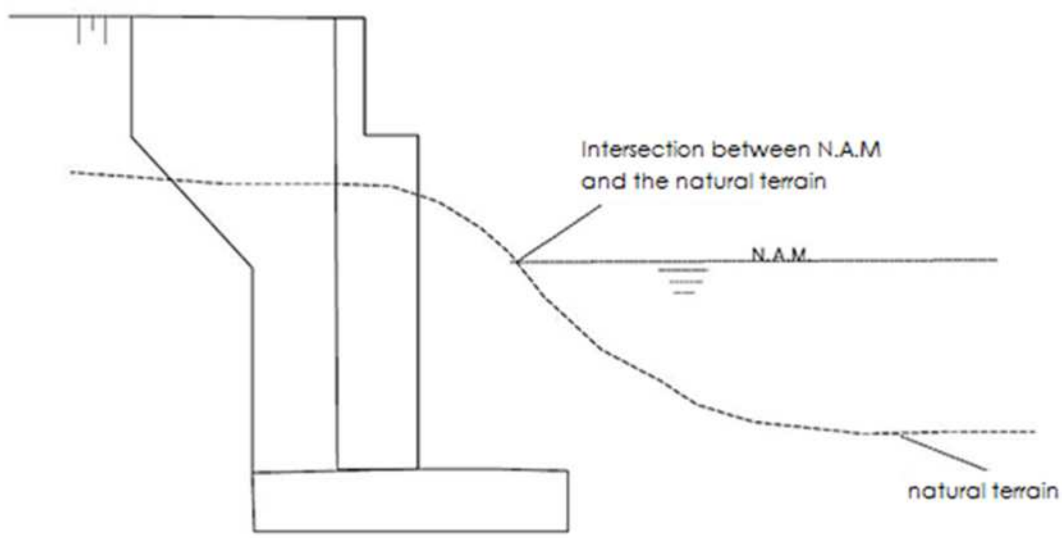


Fig. 5-17 Location of bridge abutments relative to the riverbanks and road batters

The design of piles should estimate the possibility of exposure of piles due to scour during a flood or street with a return period of 100 years (Q_{100}). It should be also reviewed other events which are believed to produce more scour.

Regardless of the type of foundation to design, we must not forget to consider the depth of scour, which must be calculated, from the design flow rate established, the flow rate recommended in the study of hydrology for the scour analysis of a specific area or under the rules established in each country.

With regard to the process of scour and its impact on the structural stability of the bridge, it should be considered the following (Flemming, 1994):

- From the geotechnical studies performed in this project, the structural analysis of the foundations should be made from the recommendations of this type of study, but in case this type of recommendations do not exist, the piers or abutments supported by piles, or with concrete supported by piles working on friction, the scour should not expose more than 50% of the pilotage, and the length without support should be less than 24 times the diameter of the strained pile situated in the set, 24 times the depth of the section for metallic piles in shape of H, or 16 times the average diameters of Wood piles (See Fig. 18).

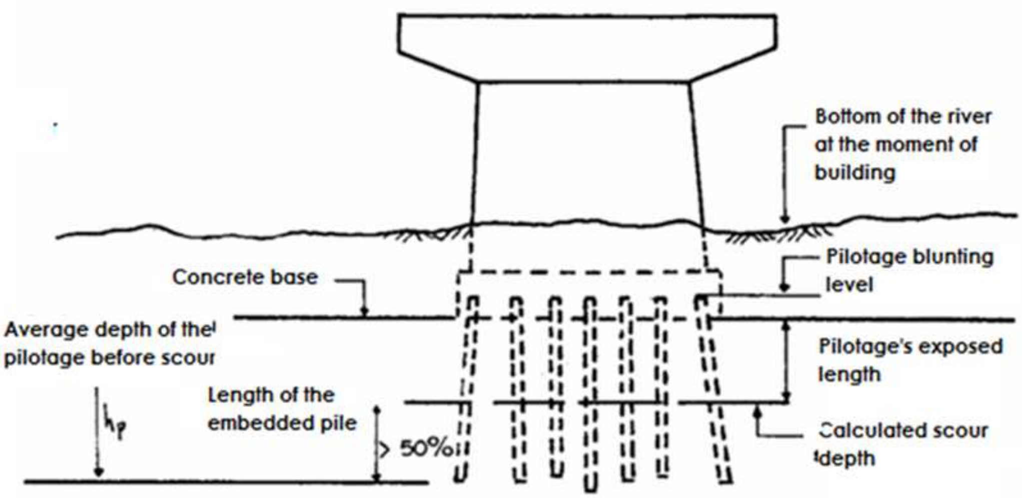


Fig. 5-18 Design of piles working for friction considering the scour effect. (Flemming, 1994)

- For piers or abutments supported by individual piles, or for piers or abutments with concrete supported by piles working by the ends, at least 1.5m (5 feet) of pile, should remain embedded in a dense material and the length without support must comply with what was exposed in the above criteria (See Fig. 19).

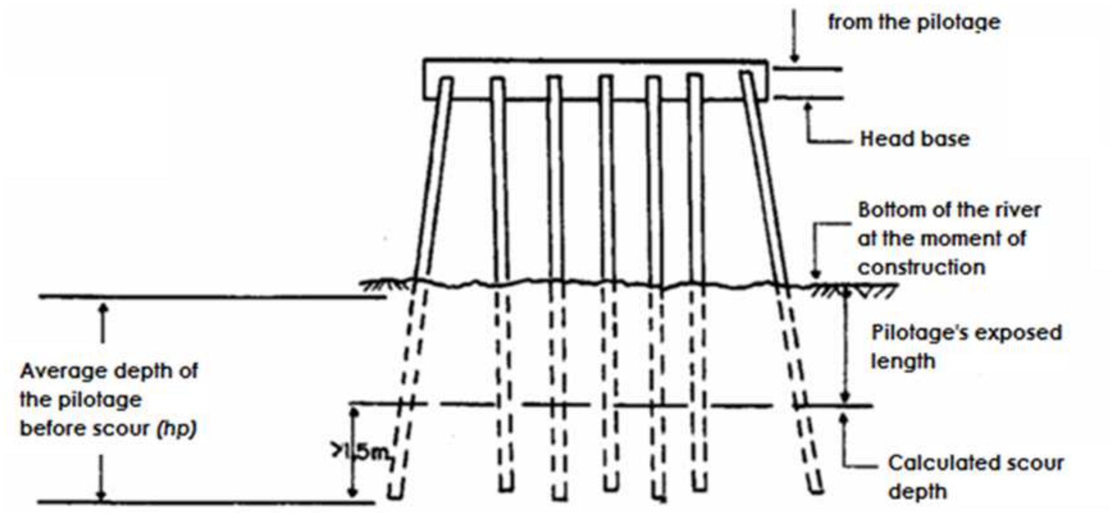


Fig. 5-19 Design of piles working by the ends, considering the scour effect. (Flemming, 1994)

The foundations of the piers in the flood zone should be designed at the same elevation of the foundations of the piers in the main riverbed (See Fig. 5-20), since there is a probability that the watercourse moves over the lifetime of the work.

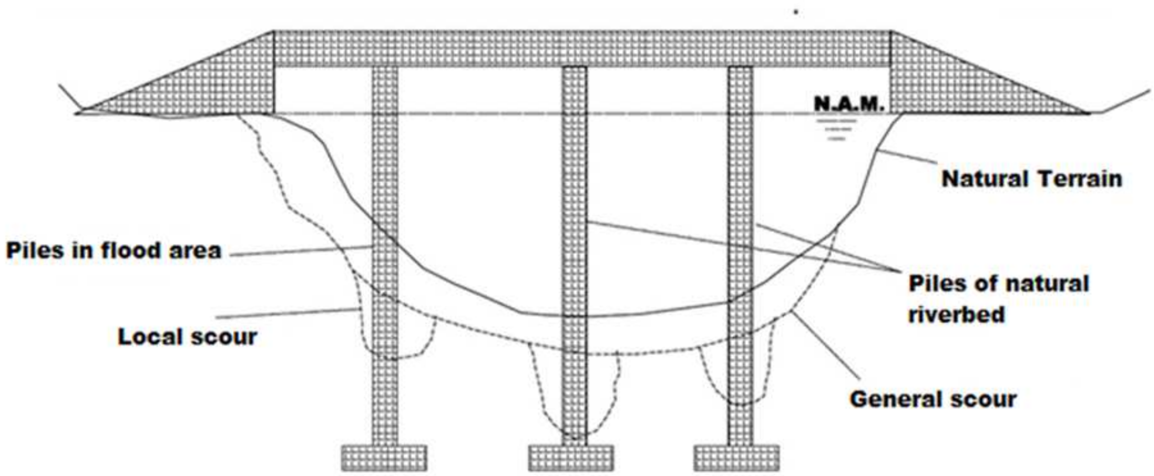


Fig. 5-20 Elevation of foundations of the piles into the main riverbed. (MOP-DACGER)

c) Hydraulic and Hydrologic Guidelines

Consider the importance of the road to define a design return period for maximum flow rates, as follows:

Illustratively it is recommended that the return period of design, according to the operational classification of the roadway may be:

- Firs Order Routes (Critical Bridges) : 200 years
- Second Order Routes (Essential Bridges) : 100 years
- Third Order Routes (Other Bridges) : 50 years

Consider free embroidery, from maximum level, resulting from hydraulic-hydrologic analysis flood. As an example, the resulting brace must be added a distance of 1.50 m for mountainous regions, and 1.00 meter for plain areas (see Fig. 5-21). The increase in the hydraulic brace obeys the excessive flow of water in rivers, transportation of debris, accumulations of deposited materials, among others. These values vary depending on the regulations of each country in the region and the characteristics of the project.

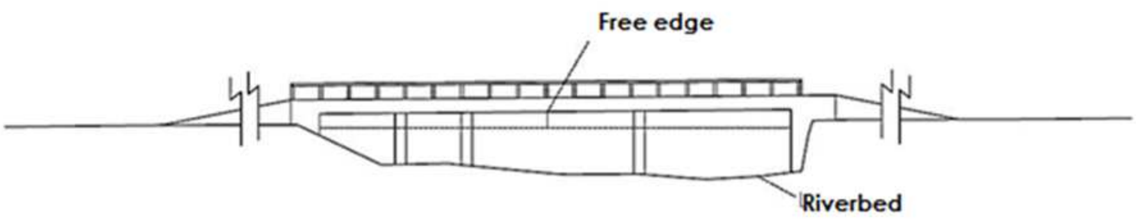


Fig. 5-21 Free brace in bridges.

Realize the analysis of the dynamics of the river, especially in the lower course, in order to determine the width of the riverbed action, its floodplain or the riverbeds of the same (paleo beds), So with this information can be defined the total length of the bridge, or otherwise to perform drainage relief (Martin Vide, 2003) at points of ancient riverbeds, or floodplains generated in the vicinity, to ease the flow thereof and reduce their accumulation. At points of old riverbeds it could be proposed a set of pipes or boxes of relief before a maximum flood that could invade these riverbeds (see At points of old riverbeds it could be proposed a set of pipes or boxes of relief before a maximum flood that could invade these riverbeds (see Fig. 5-22).

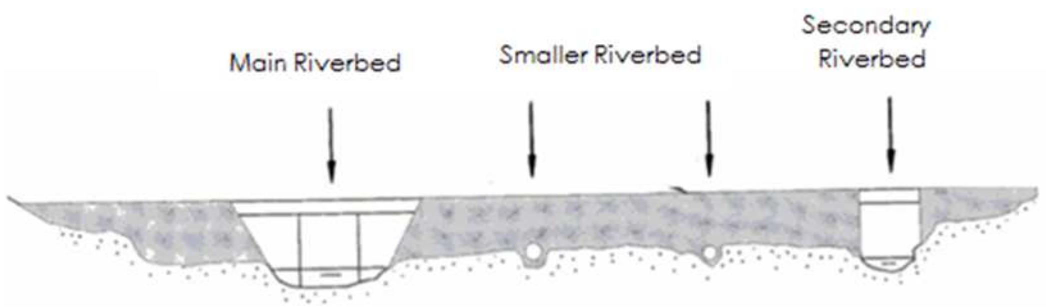


Fig. 5-22 Cross section of spans of relief (Martin Vide, 2003).

Studying the behavior of river flow should be considered in the areas near the expected proposed site, this in order to propose protection works for bridge abutments and approximations to flow impact in cases that merit, protection should be from the sides of the abutments following on the banks of the river, a distance of at least 0.5 times the total length of the bridge, both upstream and downstream. Protection can be made of gabion walls, or any other system that provides protection conditions required by type and flow direction.

For abutments and buttresses should be considered works of protection and drainage to channel the water coming from the roads to the bridge. Sometimes it has been observed that roads lack of piping works of rainwater, so it is necessary to provide these to the appropriate drainage.

Consider restriction zones (free of harmful vegetation or urban settlements) in the area of construction of the bridge. These areas must be defined for:

- The river Banks and abutments, and
- From the riverbanks within the surrounding lands (both upstream and downstream from the bridge).

Depending on the arrangement of the buttresses on abutments, to the sides thereof may be left a restriction area 2 times the width of the bridge on either side thereof or 1.5 v times the width of the bridge plus buttresses in their abutments (whichever is greater), and to the river banks may be left a distance of at least 2 times the total length of the bridge to within the lands.

Propose a program of cleaning, evacuation of materials and debris of rivers after an extreme event as well as maintain a similar program during the dry season. This will require monitoring of those bridges where the rivers are susceptible to drag sediment and debris. In those bridges, which have been built walls that keep downstream level, which provide drag material of protection for bridge piers and abutments, the cleaning program must take care not to remove too much material, maintaining a level of sediment to the crown of the walls that keep the level. (With regard to this issue, it is recommended to consult the Central American for Road Maintenance by SIECA)

Finally, in all cases, the length of the bridge must be equal or greater than the riverbed to avoid the problems of scour shrinkage.

d) Hydraulic design considerations (Ministry of Transport and Communications, Peru, 2008)

Before introducing this item, it is important to emphasize the importance of all hydrologic data related to this topic, where one of the important aspects is to obtain the design flow rate to be used and recommendations on the part of experts in hydrology make to these cases, to have a reference in this regard in chapter 4 of this document mention is made to various aspects and calculations to take into account before starting the hydraulic design of any work.

In this item will proceed to describe the general considerations for the development of fluvial hydraulics studies of bridges over natural riverbeds. It also describes in general terms the most appropriate techniques for hydraulic design and basic information for the production of hydraulic parameters.

It should be noted that good hydraulic performance not only depends on a correct analysis and proper use of the corresponding mathematical formulas; but also from knowledge of local hydraulic conditions in which its design is based on.

i. Sampling and characterization of the riverbed material

The purpose of sampling and characterization of the riverbed material is to determine the representative size that covers the entire spectrum of sizes present in it.

The riverbed material sampling must be representative, to determine its specific severity and particle size analysis. Samples of the riverbed material should be taken at least four points, two on the axis of the bridge, and 0.5B and B, where B is the average width of the river. At each point, three samples should be taken: on the surface, 1.5 times the average brace of the river, and to an intermediate depth, as long as the conditions of excavation and the presence of water allow it.

The choice of representative size for calculating scour in natural riverbeds is usually performed as follows:

- By obtaining the D of the entire particle size distribution, usually considered as the representative diameter of the entire distribution.
- The average diameter distribution is also used by the following relationship:

$$D_m = \frac{\sum_{i=1}^{n} D_i \Delta p_i}{100} \qquad (5\text{-}17)$$

Where:
D_i, is the particle size in which i (%) indicates the weight percentage of the fractions of particles which size is less than or equal to the diameter D_i.
Δp_i, is the weight percent of the material which size falls within the range which class mark is D_i, for i = 1…n intervals.

ii. Roughness coefficient of natural riverbeds (n from Manning)

To obtain the coefficient of Manning, it requires specialist expertise to make estimates, which may be based on background of similar cases, tables and technical publications available based on data collected at the stage of field.

In this item, are disclosed practical recommendations to estimate the roughness coefficient in natural riverbeds and are described below.

- In Table 5-1, are presented, as an illustrative reference, values of roughness coefficient of Manning where the value of roughness coefficient depends on various factors associated with vegetation, geomorphology and geometric characteristics specific to natural riverbeds.
- Cowan proposes a method, in which the calculation of roughness coefficient can be estimated by the following relationship:

$$n = m_s(n_0 + n_1 + n_2 + n_4) \qquad (5\text{-}18)$$

Where:
n_0, roughness base for a straight, uniform, prismatic and with homogeneous roughness canal.
n_1, additional roughness due to surface irregularities of the wetted perimeter throughout the section under consideration.

n_2, additional equivalent roughness due to variation in shape and dimensions of the sections throughout the stretch under study.
n_3, equivalent roughness due to existing obstructions in the riverbed.
n_4, additional equivalent roughness due to presence of vegetation.
m_5, correction factor to incorporate the effect of sinuosity of the riverbed or presence of meanders.

Table 5-12 shows the corresponding values to the variables used by Cowan.

Tabla 5-12 Cowan table to determine the influence of various factors on the coefficient n. (Ministry of Transport and Communications, Peru, 2008)

CANAL CONDITIONS		VALUES	
Material involved	Land	n_0	0.020
	Cut in rock		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of the cross section	Gradual	n_2	0.000
	Occasionally Alternating		0.050
	Frequently Alternating		0.010-0.015
Relative effect of obstructions	Insignificant	n_3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n_4	0.005-0.010
	Average		0.010-0.025
	High		0.025-0.050
	Very high		0.050-0.100
Degree of effects by meander	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

When beds in natural riverbeds consist of stony material where sediment is represented by an average diameter, it is recommended to use the equation of Strickel for estimation of n_0.

$$n_0 = 0.038 D^{\frac{1}{6}} \qquad (5\text{-}19)$$

Where:
D, d diameter representative of surface roughness, m.

The diameter D is equivalent to the diameter D65, D90 or D95 depending on the riverbed armoring. Particularly when the foundations provide a thick and extended particle size, the average diameter of the shell is close to D90 or D95 obtained from the original bed particle size curve.

Within the technical literature, there is the Water Supply Paper 1949 from the US Geological Survey, which has pictures of different natural flows, indicating for each case the

value of Manning roughness coefficient, calibrated with field measurements. This publication is a good reference and guide for estimating roughness coefficients in natural riverbeds.
The recommendations presented in the above paragraphs permit estimation of the roughness coefficient assuming the natural riverbed presents a homogeneous roughness, However, in nature, natural riverbeds have cross-sections that do not have uniform or homogeneous roughness, providing a composite roughness.

When the global roughness or roughness composite of the section varies with water brace, it is due that at various depths, areas of the section of different roughness are involved. This is the case of natural courses where the bed is made from a certain material and banks from other, usually in the presence of vegetation on the floodplain.

To evaluate the composite roughness, it is proposed the method of Einstein and Banks, who demonstrated by experiments, that roughness values are associated with various systems independent from each other and may overlap linearly. That is, the area of the cross section is separable from the natural course and is supposed that for each subsection is valid the Manning equation and the average velocity is uniform in the section. Then the overall coefficient of roughness generated subsystems m is given by:

$$n_c = \left[\frac{\sum_{i=1}^{m} n_i^{\frac{3}{2}} x_i}{x}\right]^{\frac{2}{3}} \qquad (5\text{-}20)$$

Where:
n_c, overall coefficient of roughness or composed of the entire section.
n_i, roughness coefficient associated with subsection i.
x_i, wetted perimeter of subsection i.
x, wetted perimeter of the entire section.
i = 1, 2 … m subsections.

e) Hydraulic calculation

The hydraulic calculation of a bridge means first determine the hydraulic capacity of the runoff section, is to say if the flow rate design properly passes through it, then determines the super elevation in water level caused by the presence of the bridge and estimate the level of total potential scour the area of the supports.

To study hydraulic capacity and calculate the super elevation of water level, a calculation at a steady gradually varied regime is performed, which allows to calculate water levels when the fluvial geometry is irregular.

The mathematical model used corresponds to a one-dimensional, not uniform, permanent and with bed fixed flow. The model is based on the implementation of the Energy Equation:

$$Z_2 + \frac{P_2}{\gamma} + \alpha_2 \frac{V_2^2}{2g} = Z_1 + \frac{P_1}{\gamma} + \alpha_1 \frac{V_1^2}{2g} + E \qquad (5\text{-}21)$$

Where:

$Z_n + P_n$, level of the water mirror at the ends of the stretch, in m.
V_n, average velocity in the wet section at the ends of the stretch, in m
α_1, $\alpha 2$, coefficient of non-uniformity of distribution of velocities in the wet section.
g, acceleration of gravity, in m/s^2.
E, total energy losses in the stretch of the watercourse considered in the calculation of a length L, in m.
γ, specific weight of water. (1000 kg/m^2).

In the above equation, the subscripts 1 and 2 refer to two different sections, section 1 located upstream section 2.

In the iterative numerical solution of the equation, the unknown is the water level $Z_1 + P_1/\gamma$ in section 1 and water level is information in section 2, $Z_2 + P_2/\gamma$. We proceed from downstream to upstream when flow is subcritical, while one proceeds inversely when the flow is supercritical.

The iterative calculation can be performed by two methods, the first method is the direct passage method and the second is the standard passage method.

A widely used model in our area is HEC –RAS (Hydrologic Engineering Center - River Analysis System), currently widely used to calculate hydraulic parameters for design cross works in natural riverbeds developed by el U.S. Army Corps of Engineers.

f) Estimate scour

Due to the scope of this document and the complexity that the issue has, scour together with the different existing methodologies, in Table 5-13, various formulas for the analysis of this parameter is presented, among which should be taken into account that many of the methodologies to estimate scour, are approximations based on experimental tests for certain types of rivers. When applied in some different characteristics, they can overestimate or underestimate the depths of scour; so it is fundamental the specialist engineer criterion. It would also be advisable to always use several methodologies for each case and compare the results, in order to obtain a reasonable estimate, according to aspects in the behavior of the riverbed that can be seen on the studied site, as well as historical records or information from neighbors. (With regard to this issue, it is recommended to consult the document "Scour of Bridges" by Dr. Maria Guevara from the University of Cauca, 2001).

Table 5-13 Methodologies for calculating scour. (Ministry of Transport and Communications, Peru, 2008)

METHOD AND DATE	EQUATION	TYPE OF SCOUR	CONSIDERATIONS
Critical velocity and clear water	$$V_{cr} = 21\left(\frac{R_h}{D_{50}}\right)^{\frac{1}{6}}\sqrt{0.056\frac{(\gamma_s-\gamma)}{\gamma}D}$$ V_{cr}, critical velocity in section, m/s. R_h, hydraulic radius in section, m. D_{50}, diameter corresponding 50%, m. D, characteristic diameter of bed, m.	General Scour	This method uses the criterion of the principle of motion of a granulated bottom under a permanent flow, same as Shields criterion and the hypothesis of clear water, ie the flow does not transport sediments.
Lischvan Lebediev	a) For granulated soils: $$H_s = \left[\frac{\alpha h^{\frac{5}{3}}}{0.68\beta\mu\varphi D_m^{0.28}}\right]^{\frac{1}{1+z}}$$ b) For cohesive soils: $$H_s = \left[\frac{\alpha h^{\frac{5}{3}}}{0.60\beta\mu\varphi \gamma_s^{1.18}}\right]^{\frac{1}{1+x}}$$ Hs-h, scour depth, m. h, water brace, m. D_m, characteristic diameter of bed, m. β, μ, φ, factors.	General scour including shrinkage by effect of the bridge.	Method proposed by Lischtvan – Lebediev, It is based on the balance to be between the real average velocity of the flow and the average erosive velocity.
Straub	$$H_s = \left(\frac{B_1}{B_2}\right)^{0.642} h_1$$ H_s-h_1, scour depth, m. B_1, width of the surface free of the riverbed upstream of the shrinkage, m. B_2, width of the surface free of the riverbed shrinkage, m. h_1, flow brace, m.	Scour by effect of shrunk section	Developed to have an estimate of the possible decrease the bed will suffer due to a reduction in cross-section.
Laursen (1995)	a) Scour due to shrinkage in the moving bed: $$\frac{H_s}{h_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}}\left(\frac{B_1}{B_2}\right)^{k_1}$$	General scour including shrinkage by effect of the bridge.	Considers the cases of scour as a result of shrinkage in the moving bed or in clear water. It is the method most used in the US (HEC-18, 1993, 1995)

METHOD AND DATE	EQUATION	TYPE OF SCOUR	CONSIDERATIONS
	H_s-h_2, average depth of scour by shrinkage, m. H_s, average flow depth, m. h_1, average depth of flow in the main riverbed upstream the bridge, m. h_2, average depth of flow in the shrunk section, m. Q_1, upstream flow rate, m3/s. Q_2, flow rate of the shrunk section, m3/s. B_1, riverbed width upstream, m. B_2, riverbed width in the shrunk section by subtracting the width of the bridge pillars, m. k_1, exponent depending on the mode of sediment transport. b) Scour by shrinkage in clear water: $$H_s = \left(\frac{0.025Q_2^2}{D_m^{2/3}B_2^2}\right)^{\frac{3}{7}}$$ D_m, effective average diameter of the riverbed material, m		
Laursen and Touch (1953, 1956)	a) Flow parallel to the main axis of the pillar: $$y_s = K_f K_g a$$ y_s, depth of local scour, m. K_f, function coefficient pillar shape. K_g, coefficient function of H_s/a. a, pillar width, m. b) Diverted flow compared to the main axis of the pillar: $$y_s = K_g K_\emptyset a$$ K_φ, coefficient function of angle of attack of the flow.	Local scour at bridge pillars	Method developed at the Institute of Hydraulics of Iowa, was developed under conditions of continuous sediment transport.
Neill (1964)	$$y_s = 1.5(a')^{0.7}h^{0.3}$$ y_s, scour depth, m. a', expected width of the pillar, m. h, brace of the upstream flow of the pillar, m.	Local scour at piles	Resulting equation of adjustment of experimental data obtained by Laursen and Toch for scour in circular and rectangular pillars.
Larras (1963)	$$y_s = 1.05Ka^{0.75}$$ y_s, scour depth, m. K, coefficient of function K_f and K_g. a, ancho del pillar, m.	Local scour at piles	Equation for estimating maximum scour depth under conditions close to the critical velocity of sediments movement.

METHOD AND DATE	EQUATION	TYPE OF SCOUR	CONSIDERATIONS
Arunachala (1965)	$y_s = 1.334q^{\frac{2}{3}}\left[1.95\left(\frac{1.334q^{\frac{2}{3}}}{a}\right)^{-\frac{1}{6}} - 1\right]$ y_s, scour depth, m. q, unit flow rate upstream of the bridge, m3/s-m. a, pillar width, m.	Local scour at bridge pillars	Modified equation from the proposed by Englis-Poona (1948)
Carsten (1966)	$y_s = 0.546a\left(\frac{N_s - 1.25}{N_s - 5.02}\right)^{\frac{5}{6}}$ y_s, scour depth, m. N_s, sediment number. a, pillar width, m.	Local scour at bridge pillars	Modified equation from the proposed by Englis – Poona (1948).
Maza-Sánchez (1968)	$y_s = H_t - H_s$ y_s, scour depth, m. H_t, scour section depth from the surface level of the flow, m. H_s, water depth upstream of the pillar before the local scour, m.	Local scour at bridge pillars	Equation applicable for beds covered by sand and gravel. The method is based on the use of elaborated curves from laboratory experimental results performed in Research Division of the School of Engineering at UNAM in Mexico.
Breusers, Nicollet and Shen (1984)	$y_s = af_1\left(\frac{V}{V_c}\right)f_2\left(\frac{h}{a}\right)f_3(forma)f_4\left(\emptyset\frac{1}{a}\right)$ y_s, scour depth, m. a, pillar width, m. V, average flow velocity, m/s. V_c, critical velocity of start of bottom particle motion, m/s. h, water brace, m. Ø, rake angle of the flow.	Local scour at bridge pillars	Equation based on experimental studies in rodded flow evaluations.
Melville and Sutherland (1988)	$y_s = aK_iK_hK_DK_\sigma K_fK_\emptyset$ y_s, scour depth, m. a, pillar width, m. K_i, correction factor of flow intensity. K_h, correction factor of flow depth. KD, correction factor of sediment size. K_σ, correction factor of sediment grade. K_f, correction factor of pillar shape. K_Φ, correction factor of flow attack angle.	Local scour at bridge pillars	The method was developed at the University of Auckland (New Zealand) and is based on envelope curve to experimental data obtained mostly laboratory tests.

METHOD AND DATE	EQUATION	TYPE OF SCOUR	CONSIDERATIONS
Froehlich (1991)	$y_s = 0.32K_f(a')^{0.62}h^{0.47}F_r^{0.22}D_{50}^{-0.09} + a$ y_s, scour depth, m. K_f, correction factor by pillar shape. a', projected width of the pillar, m. a, pillar width, m. h, flow depth upstream the pillar, m. F_r, Froude number, upstream the pillar. D_{50}, diameter of the bed particle, m.	Local scour at bridge pillars	Equation developed by Dr. David Froehlich is used by the program HEC-RAS (1998) as an alternative to the equation of the Colorado State University (CSU).
CSU	$\frac{y_s}{h} = 2.0K_fK_\phi K_cK_a\left(\frac{h}{a}\right)^{0.65}F_r^{0.43}$ y_s, scour depth, m. h, flow depth upstream the pillar, m. K_f, correction factor of pillar shape. K_Φ, correction factor of attack angle of the flow. K_c, correction factor of bed shape. K_a, correction factor of the riverbed armoring. a, pillar width, m. F_r, Froude number, upstream the pillar.	Scour at bridge pillars	Equation developed by the Colorado State University (CSU) for calculating the local scour in pillars from both, clear water and in a moving bed. This equation was developed based on dimensional analysis of the parameters affecting scour and analysis of laboratory data. It is the most used method in the US (HEC-18, 1993, 1995) and it is one of two used by the program HEC-RAS (1998).
Liu, Chang and Skinner	$\frac{y_s}{h} = K_f\left(\frac{L}{h}\right)^{0.4}F_r^{0.33}$ y_s, scour depth, m. h, depth of flow in the main riverbed, m. L, abutment length and approaches to the bridge opposing the passage of water, m. F_r, Froude number, upstream the pillar. K_f, correction coefficient of abutment shape.	Local Scour at abutments	The method is based on an equation resulting from laboratory studies and dimensional analysis, conducted in 1961. Considers a moving bed scour. Abutments are projected into the main riverbed. There is no flow in floodplain. Subcritical flow. Sandy riverbed.
Artamonov	$H_T = K_\theta K_Q K_m h$	Local Scour at abutments	Equation that allows determine not only the scour depth occurring at the bottom of the

METHOD AND DATE	EQUATION	TYPE OF SCOUR	CONSIDERATIONS
	HT , water depth at the bottom of abutment. K_θ, coefficient function of angle that makes the flow to the lengthwise axis of the bridge. K_Q, coefficient function of the ratio of expenses. K_m, coefficient function of batter to the sides of the abutment. h, water brace in the area near the abutment.		abutments, but also at the bottom of embankments or breakwaters.
Laursen	a) Scour of moving bed: $\frac{L}{h} = 2.75\frac{y_s}{h}\left[\left(\frac{y_s}{11.5h}+1\right)^{1.7} - 1\right]$ b) Scour in clear water: c) $\frac{L}{h} = 2.75\frac{y_s}{h}\left[\left(\frac{\left(\frac{y_s}{11.5h}+1\right)^{7/6}}{\left(\frac{\tau}{\tau_c}\right)^{0.5}} - 1\right)\right]$ y_s, scour depth, m. h, upstream flow depth in the main riverbed L, abutment length and approaches to the bridge blocking the passage of water. ζ, shear in the bed upstream the abutment. $ζ_c$, critical shear.	Local Scour at abutments	Equation based on the reasoning about the change in transport relations due to the acceleration of flow caused by the stirrup, for scour in a moving bed and clear water. Among the considerations there are: - Abutments projected into the main riverbed. - Abutments with vertical wall. - There is no flow on the floodplains.
Froehlich	Scour at clean water and in a moving bed $\frac{y_s}{h_e} = 2.27K_fK_\theta\left(\frac{L}{h_e}\right)^{0.43} F_{re}^{0.61} + 1$ y_s, scour depth, m. h_e, Average flow depth in the flood area obstructed by the abutment upstream the bridge, m. K_f, correction coefficient of abutment shape. K_θ, coefficient function of attack angle. L, abutment length and approaches to the bridge opposing the passage of water, m. F_{re}, Froude number in the approach section obstructed by the abutment.	Local Scour at abutments	Equation based on dimensional analysis and regression analysis of laboratory for scour in a moving bed and clear water, for abutments projected into the riverbed or not, and to concentrate flow in the main riverbed or combined with flow floodplains.

METHOD AND DATE	EQUATION	TYPE OF SCOUR	CONSIDERATIONS
Hire (1993)	$y_s = 4h\left(\frac{K_f}{0.55}\right)K_\theta F_r^{0.33}$ y_s, scour depth, m. h, Flow depth upstream the main riverbed, m. F_r, Froude number based on velocity and depth at the bottom just upstream the abutment. K_f, correction coefficient of abutment shape. K_θ, coefficient function of attack angle.	Local Scour at abutments	Equation developed from data obtained from another equation or the US ARMY for scour occurring in the tips of breakwaters built on the Mississippi River.

Besides, it is important to nothe that the method to apply will depend on the riverbed conditions, determined in the geomorphological study and the data obtained in the geotechnical study; as well as the specialist's expertise in fluvial or bridges hydraulics or from the experience in other projects located in the immediate area, preferably in the same riverbed, upstream or downstream. To calibrate the result of the applied method, it is recommended to exploit the foundation of other structures located on the same riverbed, which have already been exposed to extraordinary floods and do not have any damage, and also those that have failed by scour.

5.2.3. Subsurface Drainage

Subsurface drainage aims to drain all water flow in the ground that may affect the road structure or any flow that due to infiltration in the batters or landfills of the road may affect their and the road stability. From this type of drain, lengthwise, transverse, horizontal drains, French, systems of vertical relief wells, record boxes and sub drain drainages are known.
Subsurface drainage system must be an integral part of the total drainage system as subsurface drains must operate in line with the surface drainage system for a general system of efficient drainage.

Subsurface drainage design should be developed as an integral part of the overall design of the road, as improper subsurface drainage may also have damaging effects on the batter stability and pavement performance. However, certain design elements of the road as the geometry and material properties are required for the sub drain design. Then, the procedure usually adopted for the design of the sub drain, is to first determine the geometric and structural requirements of the road, and then subject them to an analysis of subsurface drain to determine the requirement of the sub drain. In some cases, the requirements of the sub drain will need some changes in the original design.

Therefore, it is important that within the studies to be performed in the road project, to contemplate from the beginning the studies needed for the definition of these works, such as expanding the hydrological and geotechnical studies to determine the subsurface flow that may affect the road or to determine if the project will have a height of level of construction which may have involvement due to the phreatic level in the area.

In order to avoid the problems caused by groundwater or infiltrated water in a road, it is necessary to project specific subsurface drainage systems, based on the following principles:

- Regarding the pavement, should facilitate the evacuation of water, that by deficiencies or limitations in surface drainage network or by the presence of cracks or joints in the pavement surface, it will infiltrate in it
- With regard to earthworks, it must be derived water sources appearing during construction or during operation of the road. Furthermore, it should bring down the phreatic level. This work is normally carried out during the construction phase to facilitate the implementation of the earthworks or to reduce the required thickness of pavement; however, dejection should also be performed during the operation stage of the road to stabilize batters and improve the bearing capacity of the subgrade.

One of the references from the sub superficial drainage system classification is the one presented by Garber and Hoel in which a classification is made into five general categories (Garber & Hoel, 2007):

a) Lengthwise Drains

Subsurface lengthwise drains generally consist of tubes placed in trenches within the pavement structure and parallel to the shaft line of the road. These drains may be used to bring down the phreatic level below the pavement structure, as shown in Fig. 5-23, or to remove water that infiltrates into the pavement structural section as shown in Fig. 5 -24. In some cases, when the phreatic level is very high and the road is very wide, it may be necessary to use more than two rows of lengthwise drains, to achieve the required reduction in the phreatic level below the pavement structure (Fig. 5-25)

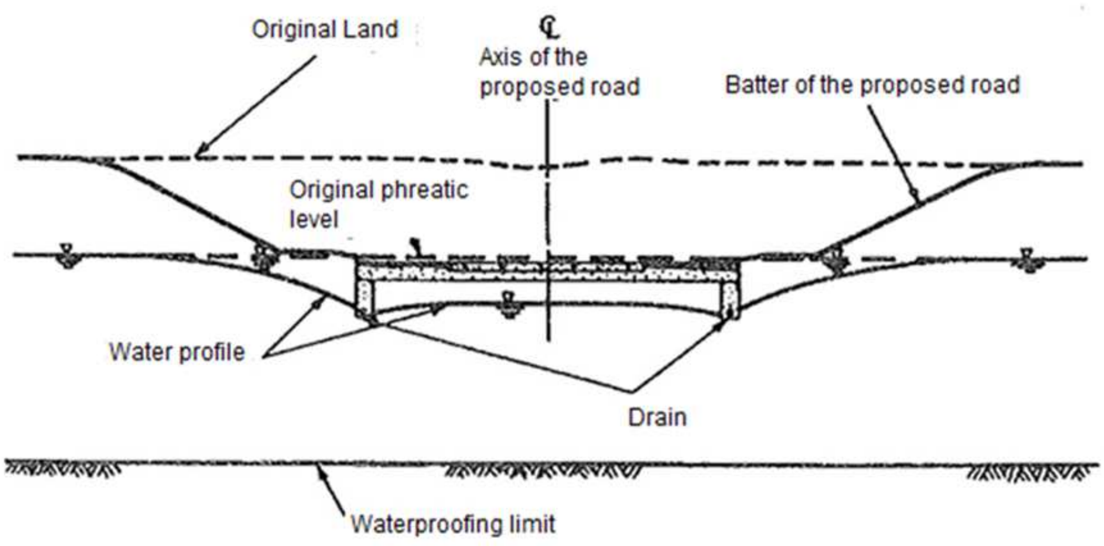


Fig. 5-23 Symmetrical lengthwise drains used to bring down the phreatic level (Garber & Hoel, 2007).

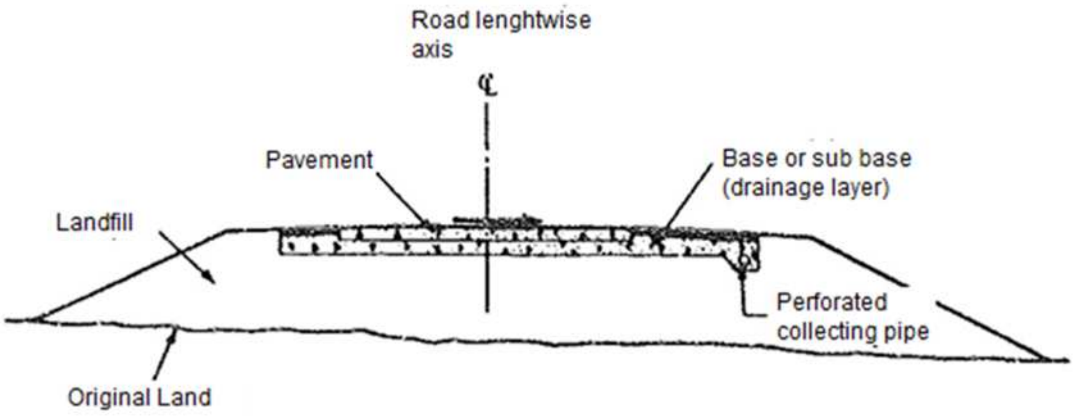


Fig. 5-24 Lengthwise collecting drain used to remove water percolating to the structural pavement section (Garber & Hoel, 2007).

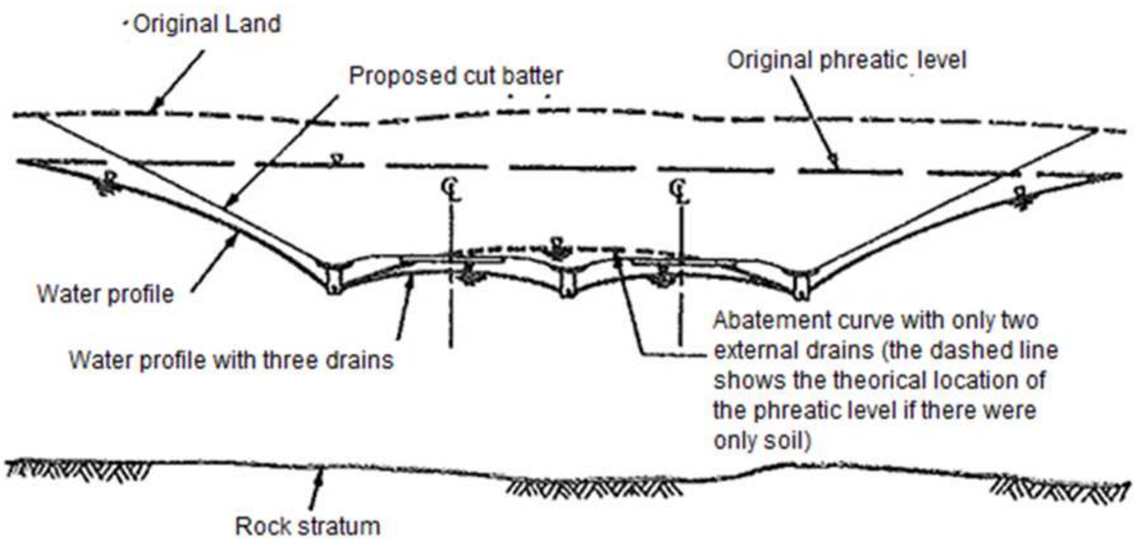


Fig. 5-25 Multiple drains and lengthwise abatement curves installation (. (Garber & Hoel, 2007)

b) Cross Drains

The cross drains are placed transversely beneath the pavement, generally perpendicular to the axis line, but may be biased to form a fishbone configuration. Fig. 5-26 shows an example of the use of cross drains, where used for draining underground water that has been infiltrated by the pavement joints.

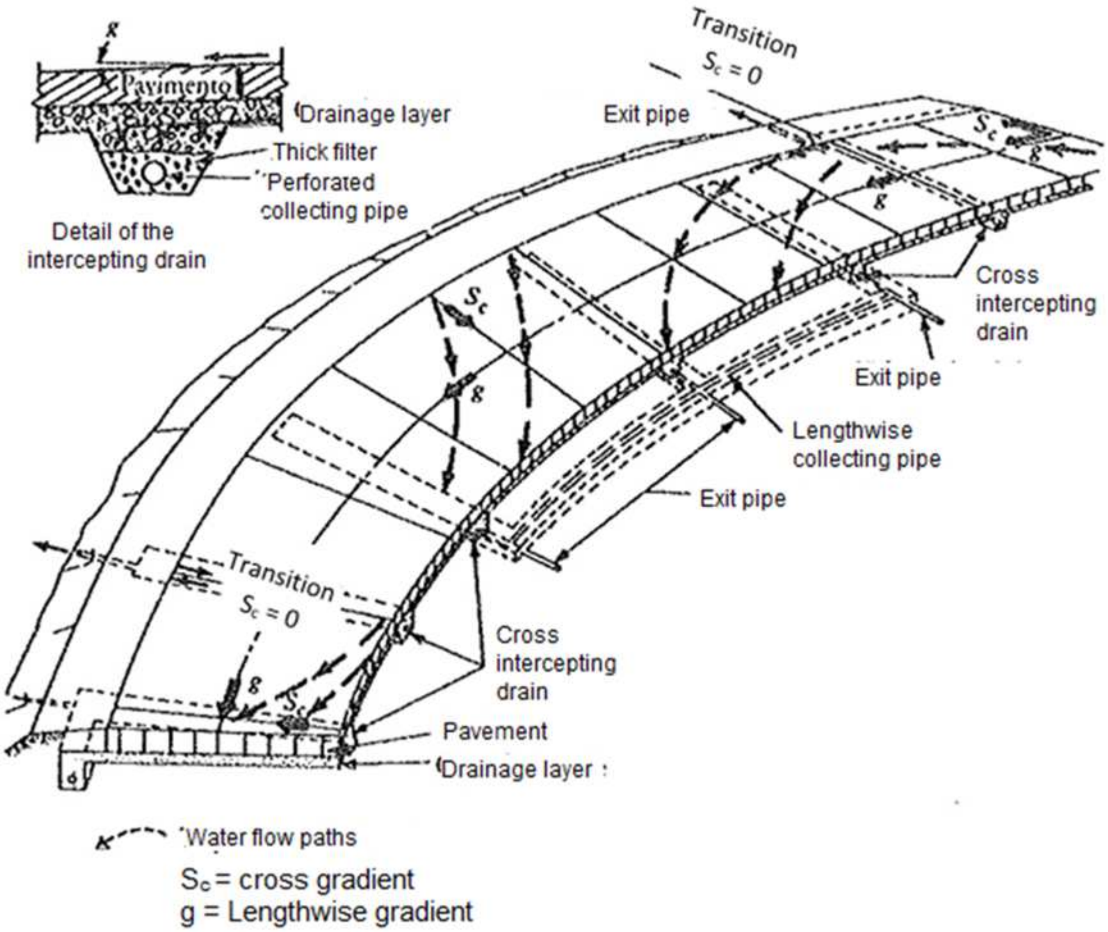


Fig. 5-26 Cross drainage in the banked corners. (Garber & Hoel, 2007)

c) Horizontal Drains

Horizontal drains are used to relieve pore pressure on the batters of cuts and landfills of the road. The drains consist of perforated pipes of small diameter, which are inserted into the batters of the cut or fill. The tubes collect subsurface water, which is then discharged in the face of the batter by covered landfills to lengthwise ditches.

d) Drainage layers

A drainage layer is a mantle of material having a very high coefficient of permeability (greater than 914.4 cm / day (30 feet / day)), and is placed below or within the pavement structure, so that its width and length in the flow direction are much larger than its thickness. The drainage layers can be used to facilitate the flow of subsurface water that has infiltrated through cracks into the pavement structure or the subsurface water from the natural watercourses. A drainage layer can also be used along with lengthwise drains to improve the batter stability by controlling the water flow, thereby preventing forming a sliding surface. In Fig. 5-27, two sets of drainage layers are shown.

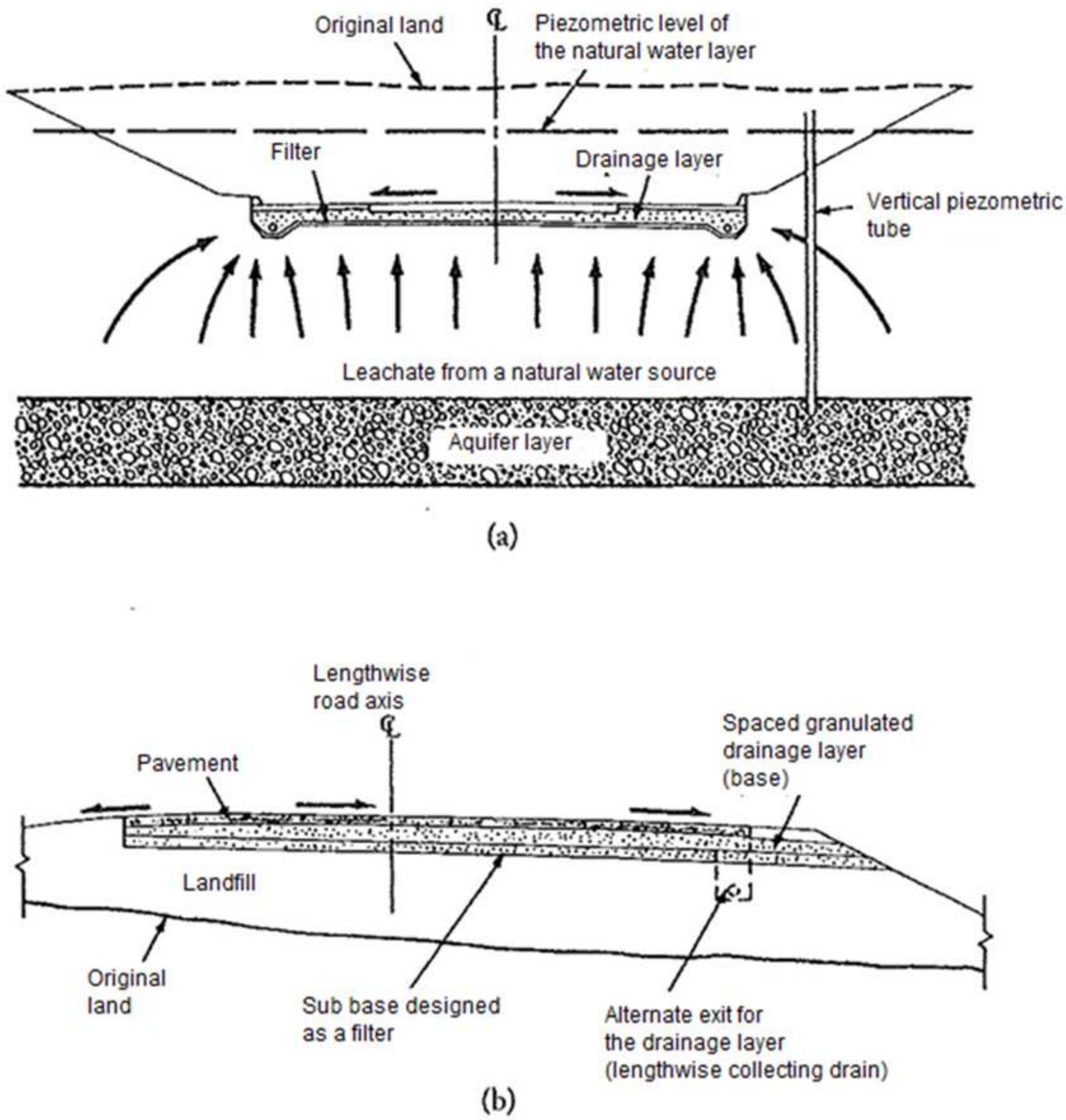


Fig. 5-27 Applications of horizontal drainage layers. (Garber & Hoel, 2007)

e) Well Systems

A system of wells is a series of vertical wells perforated into the ground, within which groundwater flows, thereby bringing down the phreatic level and relieving pore pressure. When used as a temporary measure for the construction, water collected in wells is continuously pumped out, or may otherwise be left as spill. However, a common construction consists of a drainage layer either at the top or at the bottom of the wells to facilitate the flow of water collected.

The analysis of all the above drain types may involve a whole chapter, where the design of each sub drain component is contemplated, but for purposes of this guide, it will describe the considerations to take into account for placing the sub drain and the flow rate calculation to vacate it, since from these data it is up to the designer the type of materials to be used and the existing technical specifications on the market.

5.2.3.1. Drains design

For the drain design, there are several components to design, so for this guide, sub drains will be performed under the design of the French filter methodology, which takes into account the infiltration flow rate and the phreatic level folding flow rate.

The total flow afferent filter is equal to:

$$Q_f = Q_{nf} + Q_{inf} \tag{5-22}$$

Where:
Q_f, flow rate afferent from the filter, cm^3/s.
Q_{nf}, flow rate by phreatic level dejection, cm^3/s.
Q_{inf}, flow rate by infiltration, cm^3/s.

Flow rate by provision of the phreatic level. The flow rate by the phreatic level dejections equals to:

$$Q_{nf} = K.i.A_a \tag{5-23}$$

Where:
K: Permeability coefficient of adjacent soil, cm/s.
A_a: Effective area afferent to the filter in the case of phreatic level dejection, cm^2
i: Hydraulic gradient, m/m.

$$i = \frac{(N_d - N_f)}{B} \tag{5-24}$$

Where:
N_d, lower limit of the filter required for the phreatic level not to exceed the subgrade.
N_f, upper limit of the phreatic level.
B, width of the road semibanca, m.

Also:

$$A_a = (N_d - N_f)L \tag{5-25}$$

Where:
L, maximum course length of the filter in the discharge point, cm.

Flow rate by infiltration. The flow rate by infiltration equals to:

$$Q_{inf} = I_r.B.L.F_{i.}.F_R \tag{5-26}$$

Where:
I_r, maximum hourly rainfall of annual rate recorded in the project area.
The range of 60 to 120 min is usually taken, and the 2-year curve is selected.

For the project area, IR takes values ranging from 90 to 36 mm/h.
B, width of the road semibanca, m.
L, drainage stretch length, cm
F_i, infiltration factor, dependent on the type of folder road.
F_R, retention factor of the granulated layer.

Total afferent flow rate to the filter. The total flow rate afferent to the filter is equal to:

$$Q_f = Q_{nf} + Q_{inf} \tag{5-27}$$

For the dimensions of the cross section, taking the total flow afferent to the filter Q_f, The following procedure is performed, taking into account the following equation:

$$Q_f = V_i A \tag{5-28}$$

Where:
Q_f, total flow rate afferent to the filter, cm^3/s.
V, flow velocity within the filter, which depends on the lengthwise gradient and size of aggregate used in the filter, cm/s.

Thus, A equals to:

$$A = {}^{Q_f}\!/_{V_i} \tag{5-29}$$

Also:

$$A = H.B' \tag{5-30}$$

$$B' = {}^{A}\!/_{H} \tag{5-31}$$

Where:
B’, filter width, m.
H, filter height, m.

5.3. GENERAL COMPONENTS OF HYDRAULIC STUDIES

Below are the minimum aspects to be taken into account in the hydraulic studies to present, considering that there will be peculiarities of each country in the specific structure for each area presented on the content recommendations.

- Gathering information
- Considerations to take into account as previous studies required: hydrology, geotechnics, geometric design, topography, socials, others
- Field inspection analysis to determine necessary points in drainage works.
- Analysis and calculation of drainage works:
- Lengthwise drainage
- Cross drainage (culverts and works of major passage)
- Underground drainage.
- Location in plans for the drainage works.
- Final conclusions and recommendations of the study.
- Appendices.

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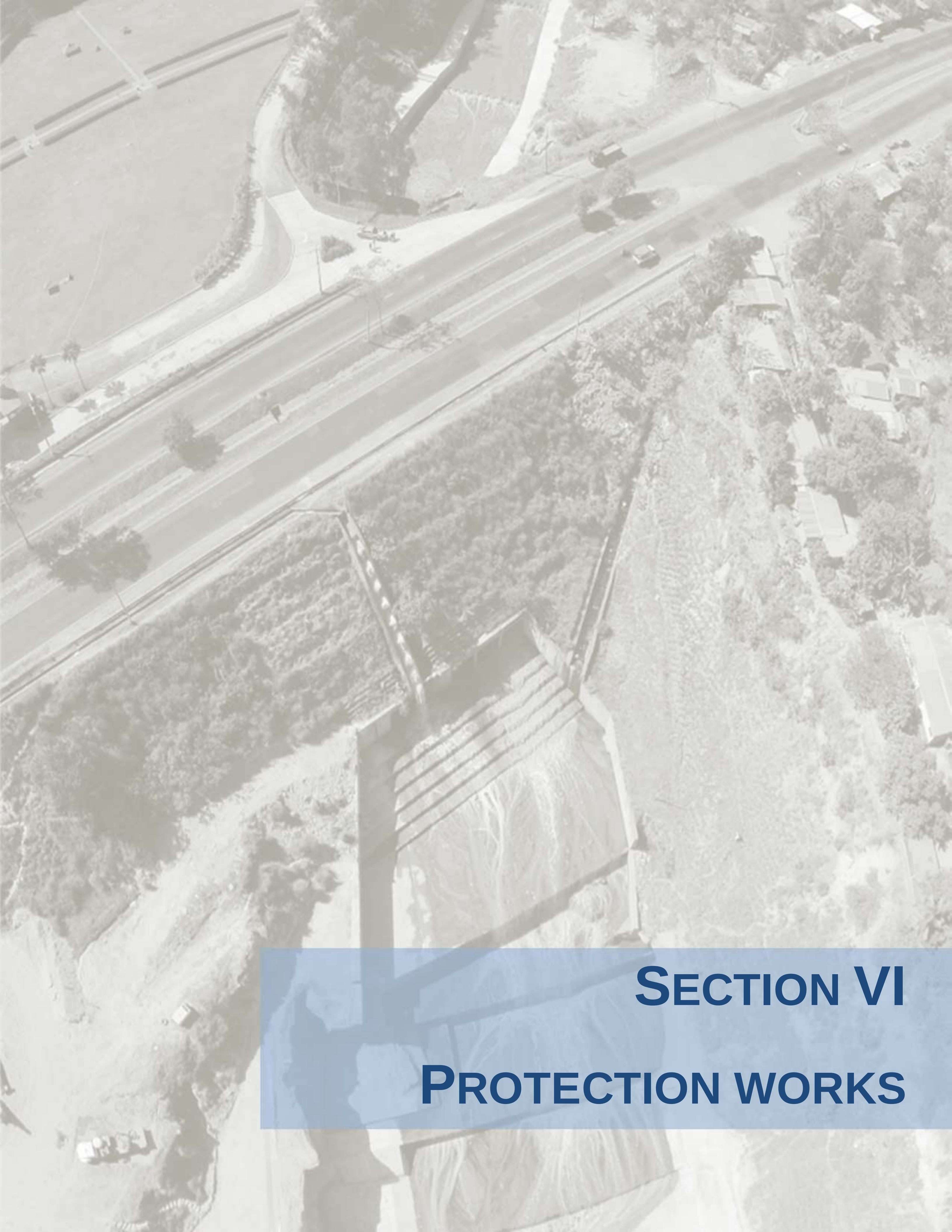

SECTION VI

PROTECTION WORKS

PROTECTION WORKS ON ROADS AND BRIDGES

6.1. INTRODUCTION

Protection works consist on structures that guarantee the lifespan of the roads, minimizing maintaining costs and procuring free circulation of vehicles and especially pedestrians, during and after possible adverse effects produced by a natural or anthropic phenomenon, and the cases that merit implementation of such structures. For the purposes of this manual, it will be approached to natural phenomena of hydro meteorological origin and structural measures aimed at prevention and risk mitigation.

It would be necessary to study if the implementation of those protection works is needed or not, depending on the conditions of existing environments where a determined project is being developed, whether new or existing, for example: land use, existence of populated areas, susceptibility degree of existing soil on hillsides, batters, river beds and others tributaries of surface runoff, among others. The consultant who designs this type of work should set conditions for implementing one or more protection works.

For this document, protective works are considered as:

- Control measures of erosion and sedimentation on roads
- Protection devices on coastal areas and shore of the lakes
- Protection works for bridges
- Fluvial protection works

Below, there are some measures considered by the consultant, as well as in bridge reparation cases due to scour.

6.2. CONTROL MEASURES OF EROSION AND SEDIMENTATION ON ROADSS (AASHTO, 2006)

The control measures of erosion and sedimentation are part of the road design, which are integrated on it to reduce the presence of a long-term sedimentation in a certain area. The timely implementation of these measures may reduce or eliminate the need for actions or temporary mitigation works. Below some measures to be considered are described.

6.2.1. Gradient

susceptibility degree of erosion of the materials involved, it can affect directly to erosion control and revegetation measures. Even though flat gradients (vertical –horizontal ration of 1:2) facilitate establishing and maintaining vegetation, this condition increases the total area of the surface that may be subject to erosion. On the other hand, experience has shown that the advantages of flat gradients outweigh the disadvantages of adding exposed areas (Fig. 6-1). Building gradients or berms is a method to break and control laminar flow in long and steep hillsides.

Grounds of slightly gradients or flat grounds allow a better compacting of filled surfaces decreasing problems of possible landslides or built up of earth for cutting sections. Rustic cutting have been used for rocky grounds, in a decomposing state or weathered rocks, in order to provide areas where vegetation can be established.

In regions with poor or infertile soils, it is recommended to gather organic soil. As well, when the embankments to one side or each side of the road are taken to their final inclination, it is recommended to put organic soil as soon as possible, spread it on the batter surface and then it can be used as a sowing bed.

Moreover, it is recommended to use synthetic materials, mostly in steep gradients cases, and according to the experience of the place, where landslides and/or loss of material of the surface of the batters face were observed because of surface runoff. Its use is also recommended to enhance vegetation growth.

Fig. 6-1 Example of embankments of each side of the road. (AASHTO, 3.5.2 Permanent Erosion and Sediment Control Measures; Chapter 3 - Erosion and Sediment Control in Highway Construction, 2006)

6.2.2. Vegetation Cover

A good vegetation cover is one of the best existing control measures (Fig. 6-2). Cushions the impact energy of raindrops and keeps the soil through the extensive root system, avoiding the loss of material. Some methods consider that planting of native grasses and woody plants, implementation of root sprouts, cuttings and grass layers.

There are various grounds where this type of protection can be implemented: grounds outside of or close to the edges of the roads, on the edges of gutters and culverts, backslopes and fillslopes, berms, among others.

There is a variety of grass, planting methods, fertilizers and fertilize processes that provide adequate vegetation cover. The institutions in charge of local agricultural extensions and services of natural resource conservation are a good source of information.

It is important to establish the correct law to conserve and keep these measures, with the efforts of the municipalities and the Ministry of Public Works in charge of main roads.

Fig. 6-2 Example of vegetated batter. (AASHTO, 3.5.2 Permanent Erosion and Sediment Control Measures; Chapter 3 - Erosion and Sediment Control in Highway Construction, 2006)

6.2.3. Open Channels.

Open, natural, or built channels are usually the most economical measure to collect and disposal of rainwater on roads, especially when the stream flow cannot be avoided.

A well-designed open channel is capable of carrying rainwater without erosion; it does not represent a hazard to traffic and provides the road of a low overall cost of building and maintenance at medium and long term. To achieve this, it is necessary to pay attention to the size of the canal, its alignment, inclination angel, protective coatings and to the structures of gradients control.

In general, the following aspects should be considered when building and designing open channels.

- **Size and Geometric Shape.** Both are important features in determining the degree of erosion of the materials that are used to build a specific canal, and/or the susceptibility to surface runoff.

  The narrow channels (also known as ditches in the region) should be adjusted to a size and shape that minimizes the impact of movement of vehicles, and providing them with adequate hydraulic section. The features used, such as low gradients, rounded transitions to obtain good security features, and are in general desirable from the point of view of potential erosion. Wide open channels will have shallow flow, and the erosive force acting on the bed of the channel is directly proportional to the flow depth. In relocation and resizing of channel, it would be necessary to pay attention to its size, stability and shape. Natural canal, if it is stable is because it has been built up over a long period of time, due to an unloading mainstream, and reaches equilibrium with minimal materials banks and scour areas.

Fig. 6- 3 Canal on the edge of a road. (AASHTO, 3.5.2 Permanent Erosion and Sediment Control Measures; Chapter 3 - Erosion and Sediment Control in Highway Construction, 2006)

- Variations in alignment of the channel, which should be gradual, specifically if channel carries fast flows. Closed curves and sudden changes of the channel should be avoided, since these conditions increase the potential of erosion towards the channlel. Most cases of channel length adjustment, either by adjustments on the alignment of a section on an existing road, a cutting in the travel distance is when curves must be eliminated. However, this is can be counterproductive, since the velocity of the flow tends to be increased; consequently this will increase the potential of erosion to the bottom of the channel. Furthermore, this change on the canal features may alter the capacity of the carrying flow sediments, to the point where the degradation and sedimentation problems will be developed within itself, among others. Therefore, care should be taken during modifications that need to be done on the alignment of an existing road, resulting as adjustments towards those drains. Mitigation of these effects can be through relocating meanders, presence of drop structures or reservoirs with structures of gradients control.

- Coatings. There are some cases when, even though, there is a good vegetation cover for the velocity control of the flow and geometric features for the scour control, among other coatings are also required. Concrete coatings, embedding material and PVC are examples of rigid coverings, which are effective as long as they are properly designed and installed. Usually the initial cost of building a rigid coating is higher than a flexible one. Maintenance costs may increase due to the susceptibility to the damage because of an excessive cutting when building the canal, and to erosion during the interface between the coating and natural surface of the canal. The surfaces of rigid coatings are in general smooth with low roughness, which is an advantage to carry the flow. However, facing scour problems due to high velocity of flow, an energy-dissipating device will be required, which can be counterproductive that the surface of these coatings is smooth. In the cases of flexible coating, breakwater, or best known in some places as “riprap”, is readily available, aesthetically pleasing and can be adjusted to changes in the base where it is used.

- Control structures. Are basically dams or spillways that are placed in a particular way that allows building open channels on gradients. In some cases, the provision of this type of structures for the erosion control at a flow rate is cheaper than the provisions of a lined channel on a ground with steep gradients. These structures are not recommended to be used for ditches, unless they are out of a pluvial collection area or if guardrails or other appropriate security barriers protect them. It must be firmly grounded on the natural ground to avoid failures on lateral cutting.

The implementation of the previous measures is recommended also in cases of gutters and approach ditches.

For more details about canals, types of sections, design and maintenance criteria, consult the following references (FHWA, Design of Roadside Channels with Flexible Lininigs., 1988), (FHWA, Design of Riprap Revetment, 1989), ((SIECA), Manual Centroamericano de Mantenimiento de Carreteras, Tomo I al III, 2010) y ((SIECA), Manual Centroamericano Normas para el Diseño Geométrico de las Carreteras Regionales, 2011) .

Fig. 6-4 Control structures in Wood canals. Extracted from (AASHTO, 3.5.2 Permanent Erosion and Sediment Control Measures; Chapter 3 - Erosion and Sediment Control in Highway Construction, 2006)

Fig. 6-5 Control structures in hydraulic concrete canals. Extracted from (AASHTO, 3.5.2 Permanent Erosion and Sediment Control Measures; Chapter 3 - Erosion and Sediment Control in Highway Construction, 2006)

6.2.4. Culverts

Culverts constrict flood flows passing through them and increase the velocity, increasing potential erosion, more than usual, in specific places. In many cases, erosion and scour in culvert crossings are harmful, either in embankments, in the same structure of culverts of downstream channels, if there is not an adequate design and protections. A good sign of needing protection on the output of culverts is the good functioning of other culverts in the area. For more details about the effects of erosion, as well as culvert design, consult the following reference (AASHTO, Chapter 4 - Hydraulic Design of Culverts, 2006).

The size of the culvert, location, skews or bias and forecast of protection in its discharge are important design considerations in determining the erosive potential of a culvert crossing point.

Generally, within an acceptable range of level, the output velocity does not vary substantially for the selection of structure sizes. However, there are cases where the water level control leads to considerable range of pipe diameters. In these cases the selection of a structure size should be done according to output velocities and where there is a potential for erosion towards the output, shall be taken appropriate protective measures. Such measures, usually, consists in reducing the velocity by means of a power dissipation device or providing a protective coating to the canal. Breakwaters rock is a good protective coating measure for canals, which also provides some energy dissipation. For more information about proposals of coating for energy dissipation, consult reference (FHWA, Hydraulic Design of Energy Dissipator for Culverts and Channels, 1988).

Culverts must also be placed along the course of natural flow if possible.

The change in gradients of a culvert should be equivalent to that of the natural canal. A thorough evaluation of alternatives for gradient change helps to identify which of these would result with less degree of erosion and scour, during and after building it. Cantilevered outputs should be avoided, unless the discharge is made towards rock strata or when other protective provisions can be taken into consideration.

Fig. 6-6 Box culvert type output. (AASHTO, 3.5.2 Permanent Erosion and Sediment Control Measures; Chapter 3 - Erosion and Sediment Control in Highway Construction, 2006)

6.2.5. Subdrainages.

The subsurface water is a common cause of landslides, unstable shoulders and other alterations of the soil contribute to the erosion problem.

Subsurface drainage systems are generally of two types: relief drainages and interception. Relief drainages are generally used to reduce the phreatic level or to help reduce the saturation of the soil and promote surface runoff, such as in basins for retaining rainwater. They are installed in a lengthwise grid pattern, draining towards the gradient or hillside.
Interception drainages are used for drain water, which infiltrate by a slope. They are installed through this and drain to one side thereof. In general it consists of a series of simple tubes, distributed and separated on a previously established by those who are in charge of the design.

For its design and location, it is necessary to identify the depth of the phreatic level or subsurface currents and dilute to know the volume of water to drain.

For more information about designing sub-drainages, refer to the reference (USDA-SCS, 1970).

6.3. PROTECTION STRUCTURES IN COASTAL AREAS AND LAKESHORES (AASHTO, 2006)

Due to projection at regional level of roads projects in coastal areas and some problems that have been occurring in existing roads, although they are not common in all countries of the region, this section is developed to serve as a reference to knowledge some solutions to these problems.

The dynamic environment in coastal areas and lakeshores often require placing some protection devices in order to ensure the stability of the road and/or major (bridges) and minor (cross drainage and lengthwise) drainage infrastructure. The structures designed to reduce the erosive effects and protect ports against the action of the waves, as well to regulate building of banks and sandbars, are classified as coastal protection works.

Some useful protection devices are proposed below, the professional who consults this document is the one who will determine the use of one or more of them, according to their needs; as well as some references are also presented for further details. It is recommended to study this reference (USACE, Shore Protection Manual, Vol. I and II, 1984).

6.3.1. Seawalls

The sewalls are essentially vertical structures built parallel to the coast, separating areas of land and water. They are designed in order to prevent erosion caused by wave action towards infrastructure nearby roads to the coast. They can also be used to protect the coast during storms, hurricanes or tropical depressions, extraordinary wave of swell or other weather phenomena related. To consider the design, refer to USACE, Design of Coastal Revetment, Seawalls and Bulkheads. EM11110-2-1614, 1995.

Fig. 6-7 Esplanade on a roadside. (AASHTO, 2006)

6.3.2. Coastal Coatings or revetments

Such coatings or revetments are structures built on the coast, parallel to it, and usually at an equal angle to the natural gradient of the coastal profile, which allows the dissipation of energy waves. They are commonly used, because they are directly in contact with the embankments of the coast. Moreover, there is a wide range of inexpensive materials available for its construction, and flexible designs for specific places.

Because of the dynamic waves toward the shore, the beach is eroded and this energy is absorbed directly on the base of the structures. In this action there is additional sediment removal and increases the height of the water from the base of the protections, increasing vulnerability to scour. Hence, measures must be taken to protect the foot of the structure.

In general there are two types of revetments, rigid and flexible ones.

a) Rigid revetments.

This type of structure provides protection against waves and currents in moderate conditions, but usually cannot stand upright in severe environments. Failures often occur if parts of the semi monolithic structure are cracked, removed or undermined. There are two special kinds of this type of coating:

- Mixture of soil and cement (soil cement), to form a moderately solid embankment; and
- Assembled structure, such as a combination of rocks connected with coating concrete and/or with concrete structures waste.

In the last case, the excavation level of its foundation should be sufficient to prevent the structure from being undermined or flanked, and should be placed in a way to not to buried after some extreme event. Rigid coatings are usually more suitable for areas with low wave activity (calm waters), as in creeks, streams and backwaters.

b) Flexible Revestments

Under conditions of light waves, flexible revetments, such as hydraulic concrete blocks, gabions, articulated coatings are best suited for coastal protection. In areas of moderate exposure, rocks are often used. All these types of coatings are able to adapt to the settlement that may occur in the foundations of major structures without generating serious flaws.

When designing protection of large areas exposed to sea or lakes, it is used with frequency embankments massive rocks. When there are no massive rocks, it could be used prefabricated coating concrete sections, as tetrapod, blocks, and other special forms, always made with coating concrete and designed for specific purposes and uses.

Protection rocks are the most common and economic used when they are good size, quality and quantity. In this regard, take care of the following aspects for this type of protection:

- Rock size;
- Excavation depth, below scour level or solid rock
- Height of breakwater excavation, to a level above the waves for splash and spray protection
- Thickness (which should be sufficient for the accommodation of rocks)
- Blanket filter, uniformly graded rock filter, geotextile filter, or both to prevent the embankment material to be displaced through the gaps between rocks;
- Foreslope, usually determined by the repose angle of embankment material, being able to use small rocks to generate mild or flat gradients.

Fig. 6-8 Rock coating. Extracted from (AASHTO, 2006)

6.3.3. Jetties

A jetty is a permeable or impermeable barrier structure, relatively thin, aligned and built to restrict or slow erosion towards the shore of the coast or to redirect natural river flows, previous research of the dynamics of the course. Basically is a spur structure that extends into the sea from the beach. The factors that depend on their design are equipment, alignment, inclination, permeability, length, spacing and configuration.

The nature of jetties is typically exposed to less severe conditions by the action of the waves, so that it extends through the surf area. Some of the materials typically used are massive stones, concrete solid blocks of great size and weight, steel plates and piles of type H sections, wooden planks and logs, among others.

In terms of alignment, this usually is normal (perpendicular) to the waterfront. The factors influencing the alignment are:

- Efficiency retaining or restricting coastal current, and
- The protection of the jetty for damage caused by the action of the waves.

The end of the jetty into the sea may have various configurations, as usually in a shape of "T", "L" or angled at the end of the section. As well, there are different types of materials for its creation: stone, wood, concrete or steel.

Moreover, the jetty permeability can be a desirable feature, whereby sediment movement downs stream is allowed.

For more information about types, materials and design of jetties, refer to (USACE, Shore Protection Manual, Vol. I and II, 1984).

Fig. 6-9 Jetty of concrete pre-built units. Extracted from (AASHTO, 2006)

6.3.4. Breakwaters

Breakwaters are parallel structures to the coastline, immersed in the sea and close to the beach and above sea level. It is often built in series parallel to the coast to form a breakwaters system. They are usually exposed to conditions of high waves and should therefore be massive structures. Breakwaters reduce the total energy when waves come onto shore of the determined coast and allow the accumulation of sediment (sand) on the projected region. Just one breakwater produces prolonged variability in wave energy, which means a winding beach of accumulation areas and alternating erosion. A wrong breakwaters system can be the result of negative effects on the flow down the coast.

It is often built with rocks, with complex made out of stones or a protective coating concrete.

In contrast to seawalls and coatings, breakwaters parallel to the coast, separated into segments, are designed to protect the coast from wave energy reflected, which leads to the accumulation of sediment and increase beach width. Only one breakwater can protect a short stretch of beach; however, a segmented breakwater system is required for longer sections of coast.

Fig. 6-10 Seawall system. Extracted from (AASHTO, 2006)

6.3.5. Bulkheads

Bulkheads are vertical structures that are mainly designed to prevent landslides; acting as retaining walls. Bulkheads are also useful to protect high areas from damages because of the waves. In coastal protection, bulkhead is a structure designed to retain the road embankment to resist the lateral soil pressure and protect it from erosion as a result of the waves.

Examples of materials used when building bulk waters:

- Concrete gravity walls,
- Wood piling, and
- Steel sheet piling with concrete screens.

The alignment of a bulkhead shall be parallel to the coastline. However, sometimes these devices are aligned with the road. The height of the bulkhead is usually set at an elevation above the average maximum level of the waves, or the average height of the waves observed following an overflow incident.

The shape of bulkheads is very important, in such a way that:

- Concentrate the power of the wave as a horizontal load on the wall;
- Diverts part of the wave that produces scour; and
- Diverts part of the wave on top of the bulk water

A check stone is often added to bulk waters to enhance its ability to prevent erosion.

6.4. PROTECTION WORKS FOR BRIDGES (GARCÍA, GUZMÁN, & PASTORA, 2015)

The dynamic of rivers and type materials of riverbeds, protection works should be considered on existing structures and in special cases of new designs, both, to supported intermediate foundations (piers) as to ends (abutments). Depending on the amount of support that the bridge has, some types of protection are proposed, first type completely or partially in the width of the riverbed, and a second type only on the margins of it.

For the first type of protection, the proposed is aimed mainly towards intermediate supports or piers, placing such protections directly on the base of it, or indirectly, by placing protection at riverbed level, both upstream as downstream.

Protection to the margins can be combined with the first type of protection, depending on the amount of intermediate supports, as well as the type of materials drag and very dynamics of river flow. Following are the types of protection described above.

6.4.1. Riverbed protection.

This type of protections are focused primarily on the piers, in case it would be implemented in all width of the riverbed, its minimum length in the direction of the river should be 3 times the width of the bridge.

To avoid the river flow going under the protection and thus causing the loss of it, it is recommendable to build protection teeth at equal or greater height of the estimated scour, both upstream as downstream. It shall be placed a Gabion cushion at the exit in order to create a transition between protection and natural land. Protection can be with a slab of concrete or masonry bound with mortar. In both cases, the upper surface must be roughened in order to decrease the flow rate under the bridge (Fig. 6-11).

If protection extends only locally in the piers, it is recommended a length in the direction of the river at least 3 times the width of the bridge, while in the direction of the bridge shall be 2.5 times the thickness of the wall of the pier from the edge of the shoe or at least 3 meters (Fig. 6-12). It will be able to implement a system of concrete slab or masonry bound with mortar leaving a rough surface. Also, scour control teeth and type gabion cushions should be placed downstream, as described for protection across the width of riverbeds.

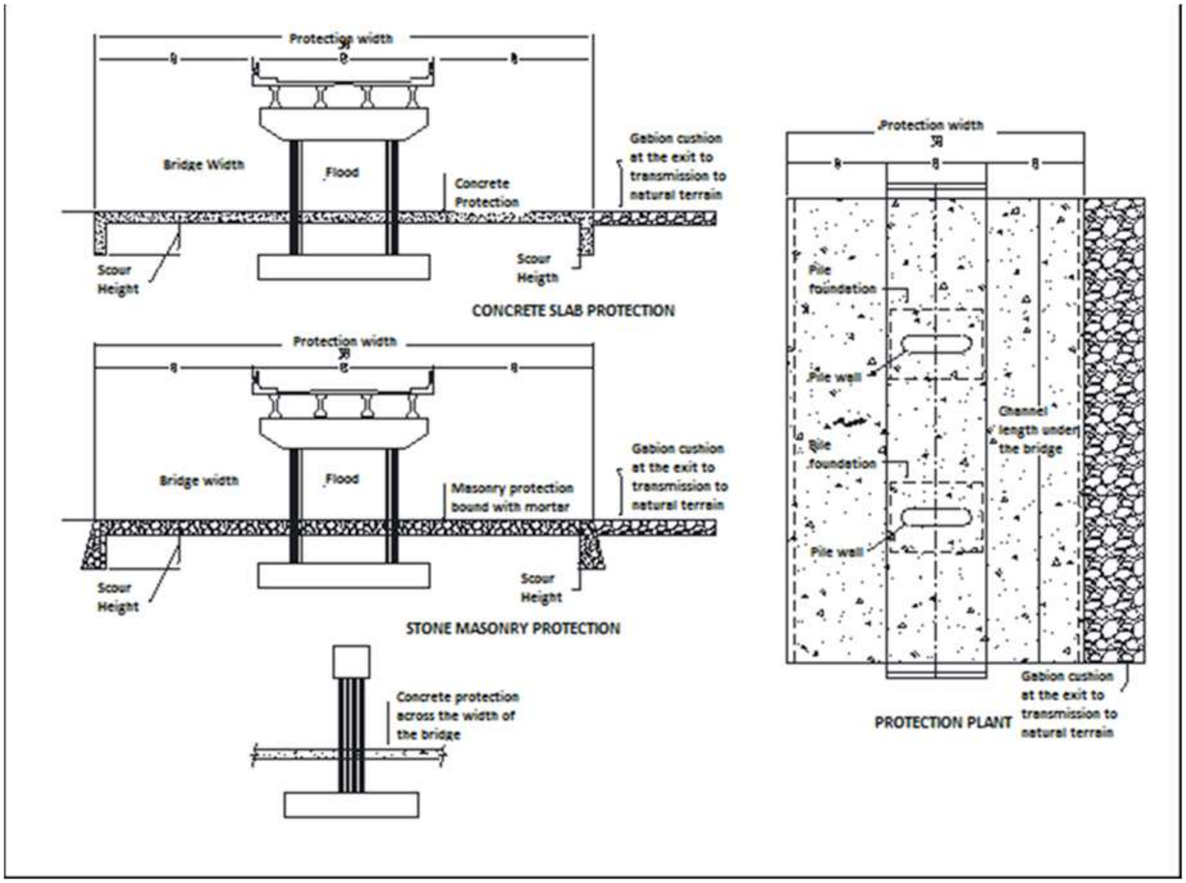

Fig. 6-11 Protection type on piles, across river bed width. (SPOP, 2014)

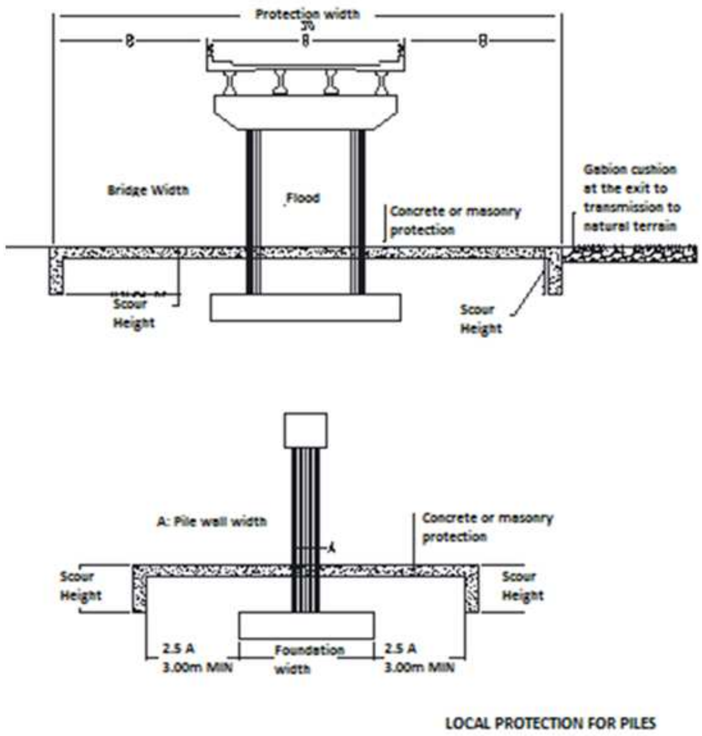

Fig. 6-12 Local protection type on piles. (SPOP, 2014)

In addition to this type of local protection, the use of prefabricated concrete blocks is proposed, and they are placed around the base of the pier that will be protected (Fig. 6-13). These elements must have such a weight that cannot be washed away by the river flow.

This technique is to minimize potential local scour processes in piers and avoiding differential settlement in their foundations. Moreover, it is proposed the use of sheet piling in the same way that the blocks, and it is recommended this technique in places where the soil type allows to drive sheet piling below the calculated scour level, so that it remains stable (Fig. 6-14). Usually the type of soil where sheet piling is recommended, it is planned for foundation of piles and abutments supported on piles, being that sheet piling protects both foundations as piles from scour.

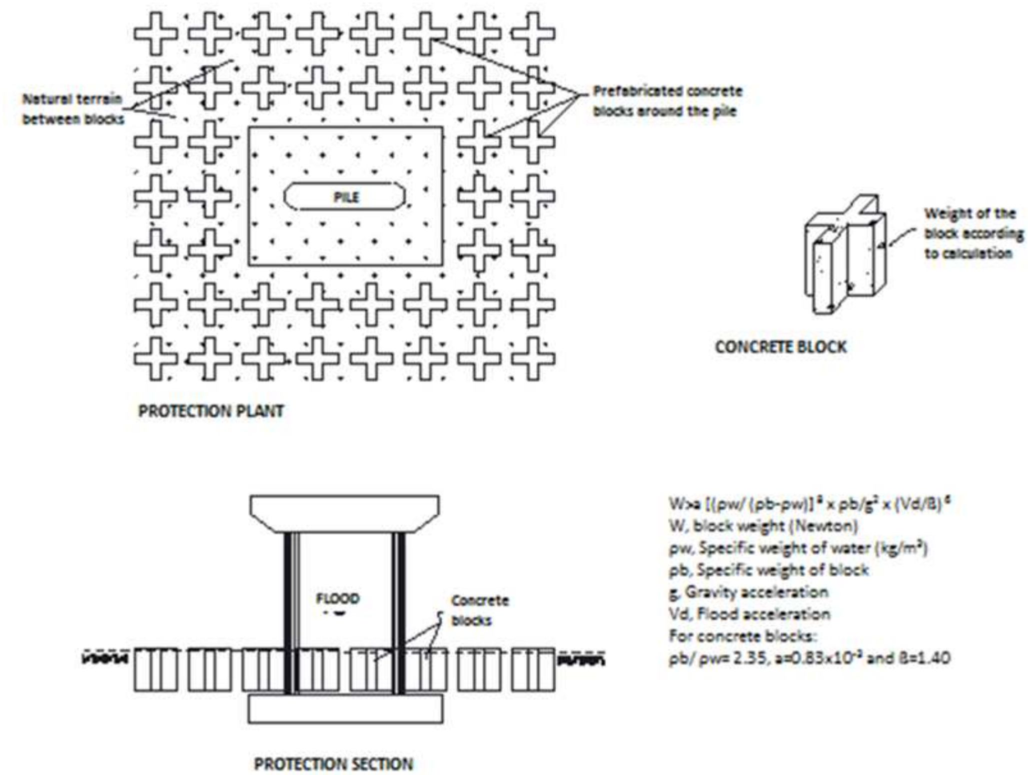

Fig. 6-13 Local protection in piles using prefabricated concrete blocks. (SPOP, 2014)

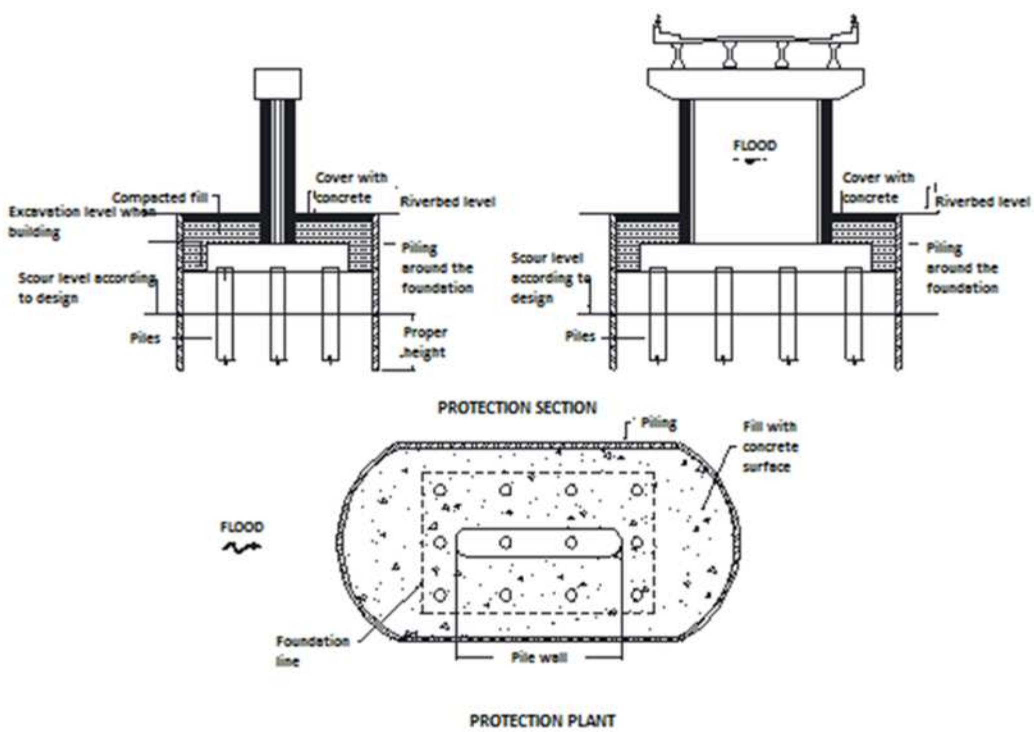

Fig. 6- 14 Local protection in piles using sheet piling. (SPOP, 2014)

An indirect way of protection for piles support on bridges is the use of grade walls, which are transversely embedded in the bottom of the riverbed and lateral to the banks, placing them downstream of the bridge. The wall must be located at a distance from the bridge and with a height according to the desired gradient, in order to retain drag material until the bottom is filled to change the gradient, lessening the velocities and when floods occur, obtaining a laminar flow (non-turbulent) in this stretch.

It should be noted that the height of the grade wall must de designed in a way not to increase the riverbed level in the bridge location. Just as in piled foundations, if this kind of protection is used on uneven ground (sand or gravel), it should be considered the implementation of piles in the foundations of these walls (Fig. 6-15).

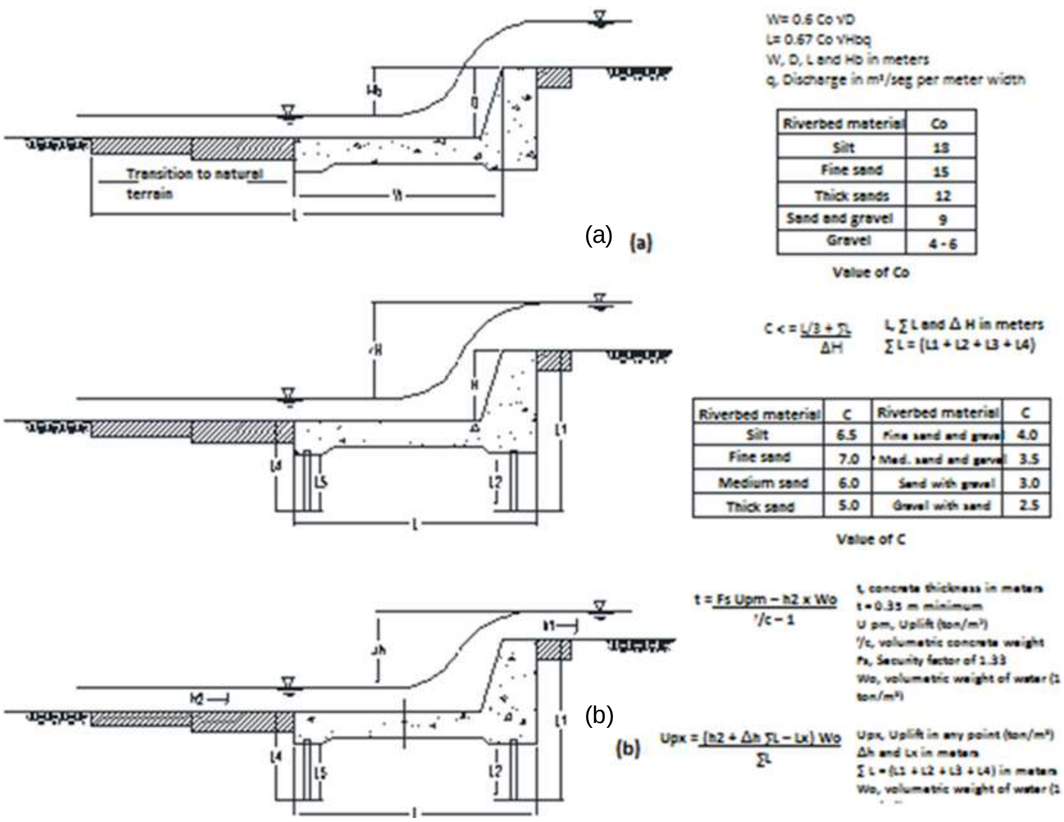

Fig. 6-15 Proposed types of indirect protections through grade walls (a)Grades for beds of firm or rocky soils; (b) Grades with piles for beds with loose soil (sand or gravel) (SPOP, 2014)

6.4.2. Protection of riverbanks.

This type of protection can be performed in combination with the protections across the width of the riverbed under the bridge, with tooth type walls at the edges upstream below the level of riverbed to counteract the effects of scour, and using gabion type cushions downstream, in transition to the natural terrain (Fig. 6-16). It is advisable to develop in the direction of flow of the river over a length of at least 3 times the width of the bridge.

Moreover, if the bridge is a single course, without intermediate supports, can be applied locally, with a length in the direction of flow of the river at least 3 times the width of the bridge and a width measured from the edge of the foundation of the abutment toward the center of the riverbed at least 3 meters (Fig. 6-17).

If it intends to form protective embankments, projected from halfway up towards the level abutment riverbed, protection to the face of gradients, which can be with concrete slab or gabion type cushion is proposed. The maximum gradient of the gradients recommended to apply this type of protection is 1: 1 (horizontal / vertical relation), being necessary to place anchors if the gradient is greater than 1.5: 1.

It is noteworthy that for the case of protection with concrete, it is necessary to place drainages to evacuate water accumulation in the gradient, not the case for gabion type cushion as the drain is through the voids between the stones (Fig. 6-18), and as in the previous cases is required to apply a control tooth scour at the bottom of the gradients, with a depth of at least 1 meter. This proposal is also feasible using stone lean concrete base, as long as drainages are placed, as in the case of the concrete slab, and a maximum batter gradient is 1.5:1 (Fig. 6-19)

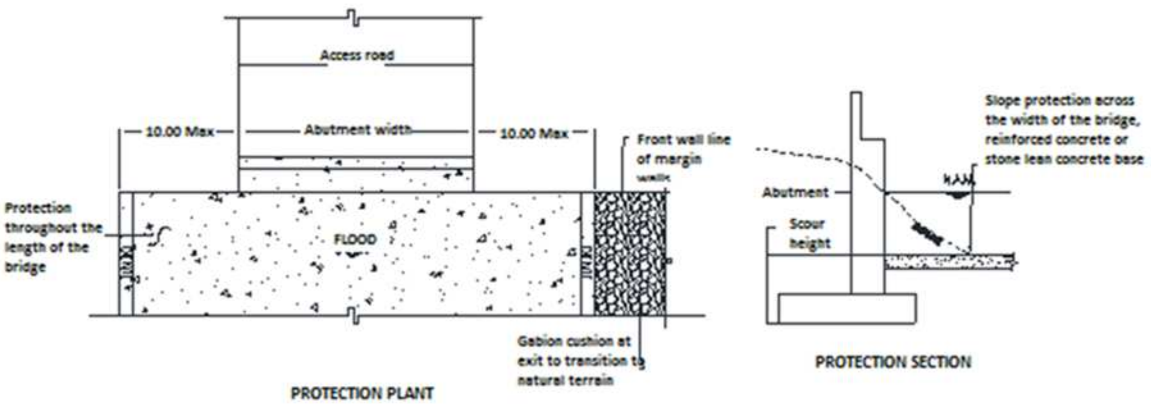

Fig. 6-16 Type of protection at the abutments for the entire width of the riverbed under the bridge. (SPOP, 2014)

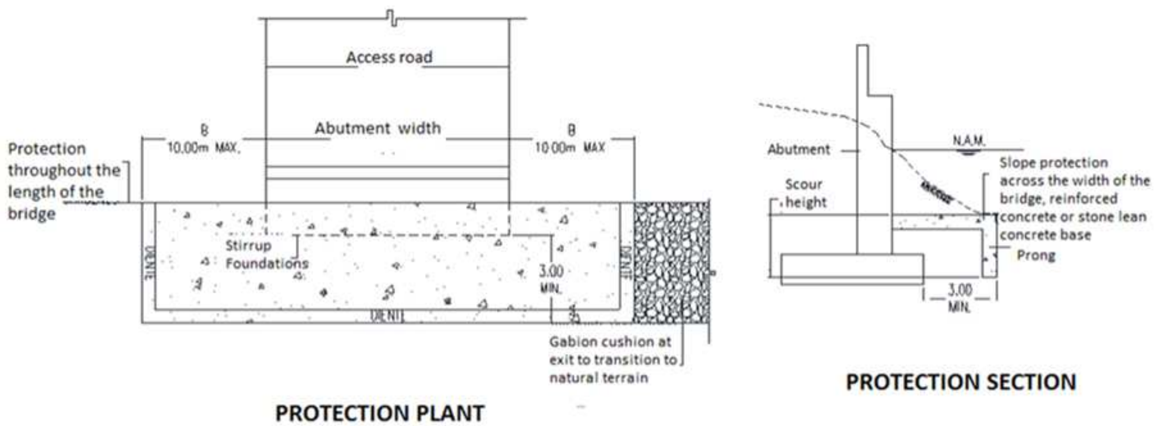

Fig. 6-17 Local protections for abutments. (SPOP, 2014)

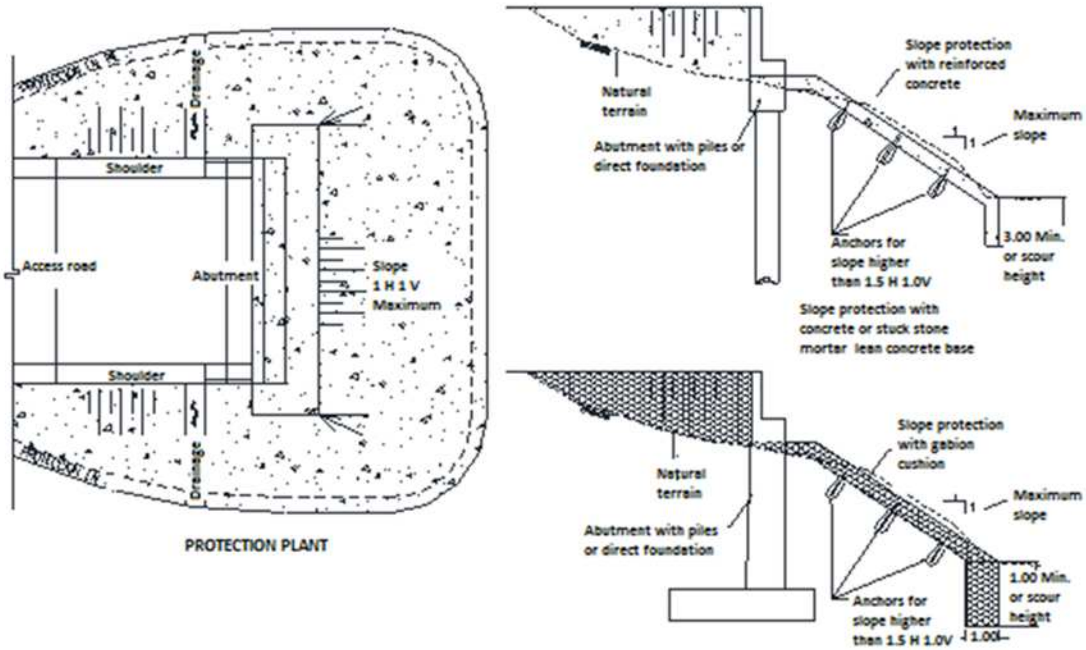

Fig. 6-18 Protection abutments without walls on the banks of the river, with anchors. (SPOP, 2014)

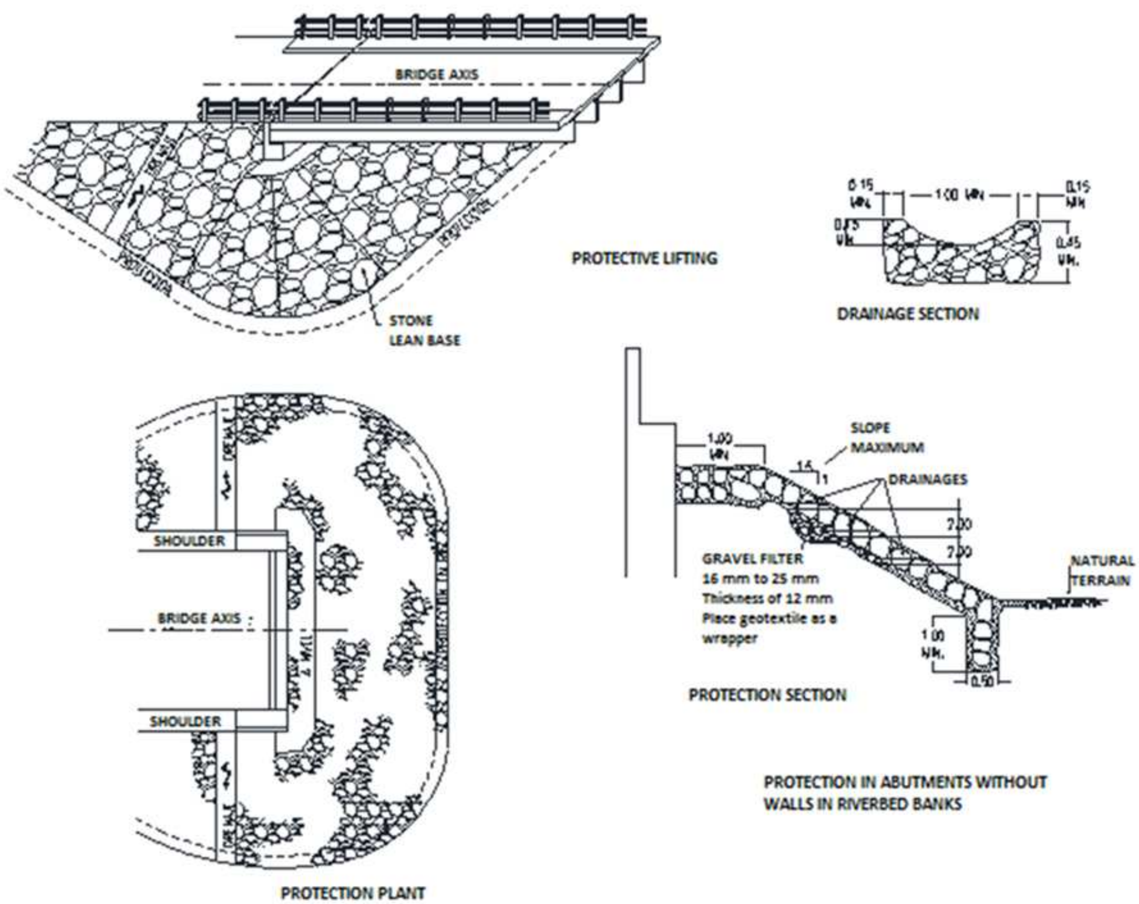

Fig. 6-19 Abutments protection without walls on the banks of the river with stone lean concrete base. (SPOP, 2014)

In the case of protective walls for banks, adjacent to the bridge abutments, to counteract the local scour at their bases, it is feasible to apply a gabion cushion as part of its foundation.

Applying this proposal, when in its implementation, can be seen in the front of the walls, in case of scour, that the cushion is becoming established by its flexibility, thus protecting the base of the wall (Fig. 6-20). This deformation is an indication that the wall must be repaired, and the materials at the cushion base that have been removed in the scouring process must be restored.

A similar proposal is to apply a slab of concrete or stone masonry (Fig. 6-21). However, because of being rigid materials, it is not recommended to use this proposal in riverbeds with very fine or granular materials, since the generated material removal from the slab base caused by scour effect, would tend to fracture it, creating spaces where part of the river flow could infiltrate below it, and reach support material to foundations of the wall, causing a resistant reduction thereof and ultimately producing differential settlements, risking a partial or total collapse of the bridge. In cases of joints between face walls and the edge of protections, as shown in Fig. 6-21, it is recommended the use of fasteners, such as anchoring hooks, so that protections have more fastening in case of an eventual extraordinary flood.

For more information about local and general protection on bridges, consult (Guevara, 2013).

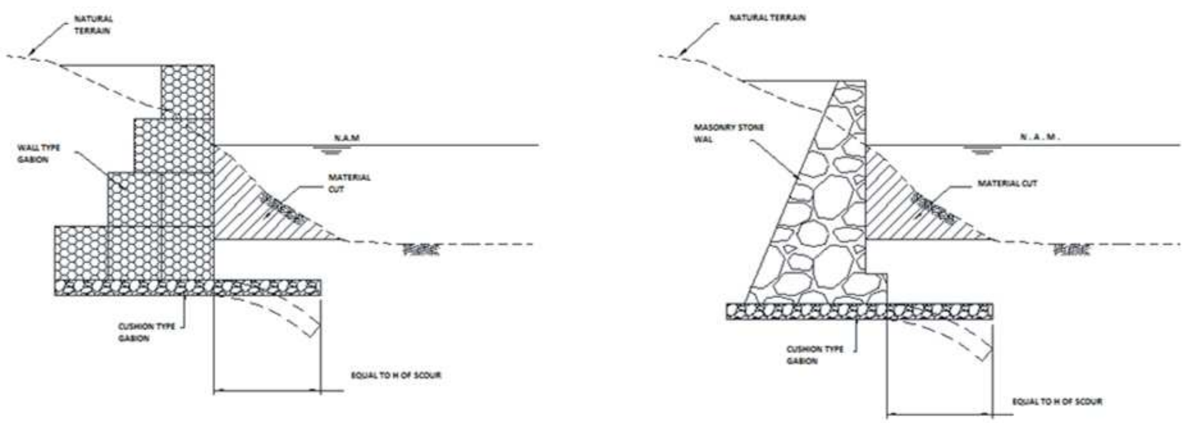

Fig. 6-20 Types of protections for bank walls with cushion type “Reno” (gabion type). (SPOP, 2014)

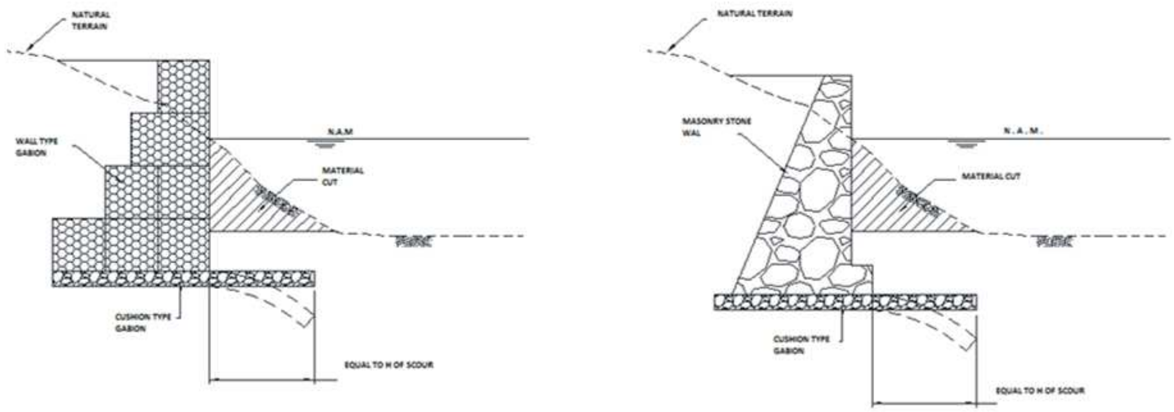

Fig. 6-21 Types of protections for bank walls with slab protection. (SPOP, 2014)

6.5. REPAIR IN EXISTING BRIDGES DUE TO SCOUR

One of the main types of damage observed in bridges, after a remarkable hydro-meteorological phenomenon, is the one caused by scour phenomenon. Hence, some repair methods of piles and abutments methods are proposed. The implementation of these proposals for repair, depends on the general condition of the bridge, after scouring occurred, as it is determined that the superstructure has been partially affected, it is recommended in most cases its total or partial replacement.

6.5.1. Repair in direct foundations of abutments and piers

In the event that scouring has occurred beneath the foundations of piers and abutments, which were built very close to the level of the riverbed, and were not supported on piles (foundation directly supported on riverbed materials), it is recommended to inject fluid mortar to reestablish 100% contact with the foundation area and thus restore the transmission of loads evenly towards the foundation soil (Fig. 6-22). The implementation of fills with plastic soil cement is not recommended, since this mixture has more susceptibility to the erosive action of the river flow than the filling with fluid mortar.

6.5.2. Repair in supported foundations in piles of abutments and piers

In case that scouring has occurred under a foundation supported on piles, and where a part of them have been uncovered, it is recommended to fill with fluid mortar to restore the lateral confinement thereof, protecting said confinement with sheet piling, when is possible to place it (Fig. 6-23). The implementation of fluid mortar and plastic soil cement is feasible, since in this case the transmission of loads is towards deeper layers, although it is more advisable to fill with the first type.

6.5.3. Repair in foundations of abutments with severe scour

Again, in the case of abutments with foundations resting on piles, when there is severe scour, that is, that all piles have been exposed due to scour, and also a portion of riverbed is lost; it is recommended to apply a combination of actions, it is proposed to drill from the back of the abutment wall at ground level of the bridge, for the implementation of fluid filling, and jointly implement the second repair method described above. It is advisable to place type gabion cushions towards the exposed face from the sheet piling, as its additional protection from river flow, and transported material or debris that may affect with the protection (Fig. 6-24).

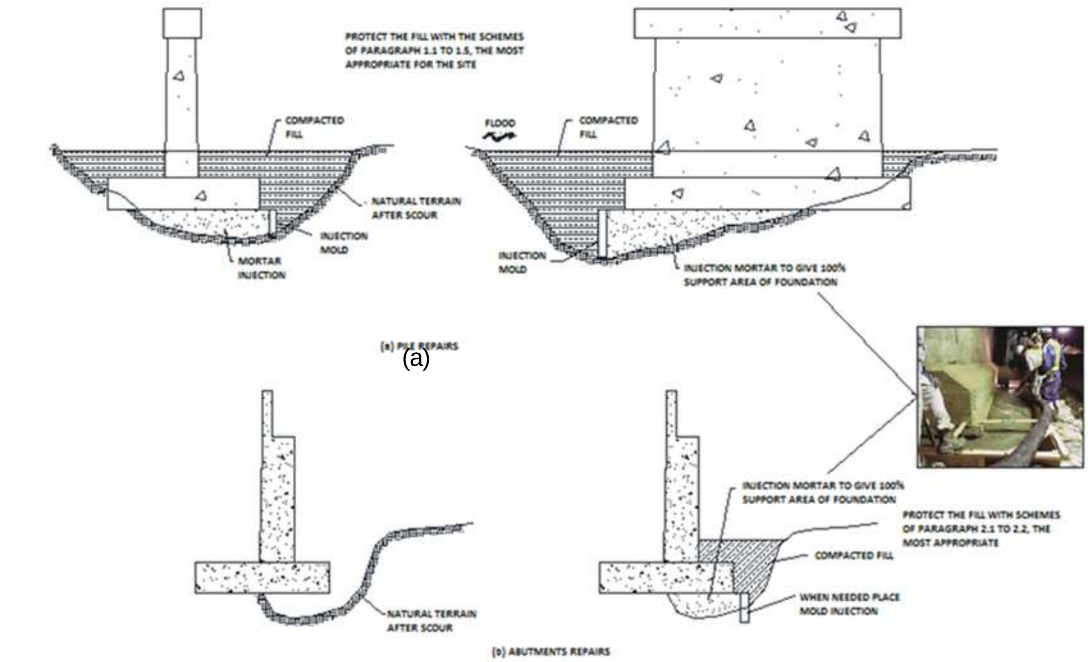

Fig. 6-22 proposed repair method for direct foundation in (a) abutments and (b) piles. (SPOP, 2014)

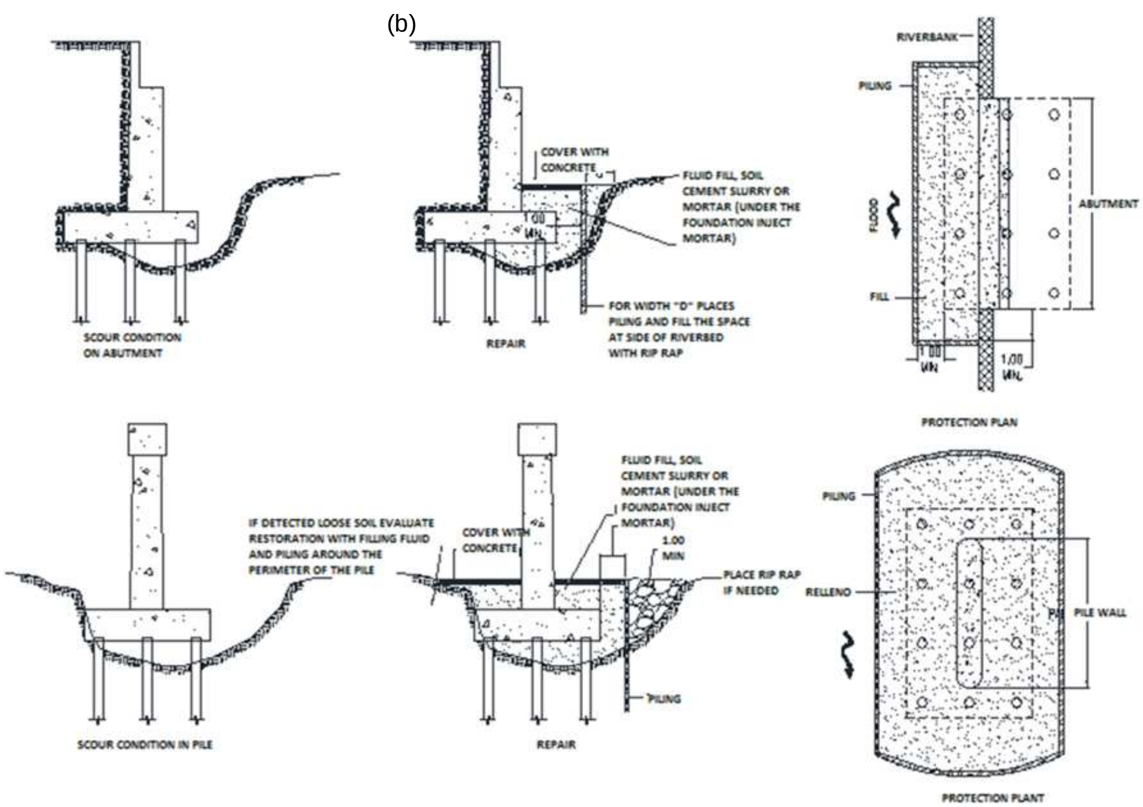

Fig. 6-23 Repair proposal in piers and abutments of foundation with piles. (SPOP, 2014)

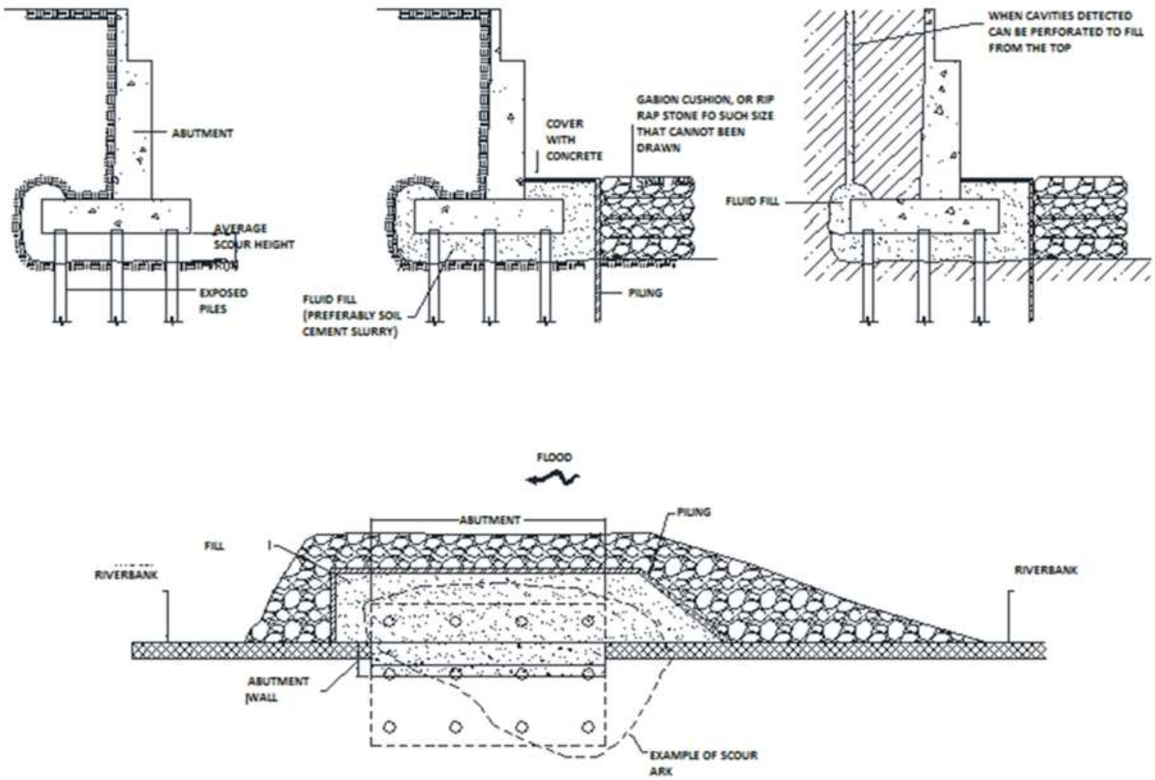

Fig. 6-24 Repair proposal on abutments with severe scour. (SPOP, 2014)

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SECTION VII

REFERENCE SOURCES

REFERENCES SOURCES AND CRITERIA TO TAKE INTO ACCOUNT IN THE HYDROLOGIC AND HYDRAULIC STUDIES BY COUNTRY

7.1 COSTA RICA

Governing institution	Ministry of Public Works and Transport (MOPT)
Members of the Regional Technical Group (RTG)	■ Antonio Romero Castro. Bridges Office. ■ Christian Fernández Camacho. Sectoral Planning Secretariat.
Technical Reference Documents	■ *Hydrologic and Hydraulic Design of Minor Drainages on Roads*. Thesis presented by Engineer Ramiro Gamboa to School of Civil Engineering at the University of Costa Rica (1969). ■ *Construction Manual for Roads, Highways and Bridges*. Prepared by the Department of Regulations, General Office of Constructions, Ministry of Public Works and Transport (1983). ■ *American Association of State Highway and Transportation Officials* (AASHTO) Highway Drainage Guidelines. ■ Publications HEC18, HEC 20 y HEC 23 of Federal Highway Administration (FHWA).
Cartographic material	The topographic sheets are available at the National Geographic Institute (IGN) at a scale of 1:10000. The National System of Territorial Information (SNIT) also exists, which is a reference tool that integrates and distributes territorial information of the country. The website of this system is **www.snitcr.go.cr** , cartographic visor section.
Road data bases	MOPT has a Bridge Management System (SAEP). Such database does not include historical records of road damage but it has records of damaged bridges, and they contemplated the damage caused by Scour
Hydrologic and hydrometric instrumentation	The Costa Rican Institute of Electricity (ICE) administers the largest and most comprehensive network of instrumentation. Also, the National Meteorological Institute (IMN) and the University of Costa Rica (UCR).
Hydrologic information available	Hydrologic records can be acquired in the ICE, IMN or UCR through formal request or agreement if public institutions, or purchased in the case of consulting firms. It should be noted, that ICE has raw data and analyzed information or sub-products. Information is strategic and used for hydroelectric purposes.
Methodology for hydrologic analysis	There is no established methodology for maximum flow calculation. Recommendations, methods and software applications listed in specific publications shall be used for calculation of peak flows in natural basins, as long as a reference to used bibliography is made. The National Highway Council of Costa Rica (CONAVI) recommends, in particular, the free access documents HEC and HDS, published by the

	Federal Highway Administration (FHWA), Department of Transportation of the United States and the National Highway Institute (NHI), available on the website **www.fhwa.dot.gov.** While it should be noted that these technical documents include issues that do not apply to Costa Rica. In the case of using the rational formula, CONAVI in its terms of reference, requests for basins with less than 2.5 km^2 (250 Ha). For MOPT, this limit can vary and be higher as long as the results are properly justified and considered valid by the institution. When using a physical or mathematical model, it must be indicated all the inlet data, type of model used and the outlet, as well as the limitations of the program. The Contract Supervisor Unit should be able to perform again all models for verification. **RUNOFF COEFFICIENTS** Geology and vegetation cover of each basin must be taken into account and analyzed for allocating a runoff coefficient to the field. The assigned value must be duly justified, indicating the process of each result. For each basin, it should be tabulated: its area, the weighted runoff coefficient, intensity and time of concentration. Recommended runoff coefficient values can be found in the document *Hydrologic and Hydraulic Design of Minor Drainages in Roads*. The design flood is determined either based on the analysis of the maximum storms in the basin or the region, or transferred to the basin under study.
Consideration due to variations in rainfall patterns in hydrologic analysis	They are not punctually specified. However, the exceptional weather records due to events such as storms or hurricanes that have been representative must be particularly considered.
Methodology for hydraulic analysis (Source CONAVI)	For each structure design should be defined an area of influence to the point where the expected work is located. In the case of pluvial culverts, each water discharge coming from other structure to the system (e.g. a storm drain inlets), should be analyzed as an additional basin that reaches the point from where the support comes. **MAJOR AND MINOR DRAINAGE** The distinction between major and minor drainages according to the design flow, is as follows: **Major drainages:** structures crossing the road over watercourses whose flow design exceeds 15 m^3/s.

Minor drainages: within this group are included storm drain inlets, pipelines, curbs and gutters and buried culverts, which have a lower flow rate to 15 m^3/s.

PREMISES FOR THE DESIGN OF DRAINAGE STRUCTURES

RETURN PERIODS

The hydraulic design of major drainages is done for peak flow estimated in the study site using a return period of 100 years, while for minor cross drainages to the road will be done for 50 years.

In the case of minor roadside drainages, it will be done using a return period of 25 years.

There is a possibility that the consultant deems appropriate to use a different value for specific analyzes or type of structure, but this should be justified, so that the Contract Supervisor Unit approves to use the proposed one.

For major drainages, should be done a structure behavior review for a peak flow of 500 years, although the design is, as indicated, for a return period of 100 years.

MAJOR DRAINAGE

Prior to the design, it must be made the following watercourse analysis of major drainages:

- Riverbed morphological analysis. Horizontal and vertical stability. Historical aerial photographs may be included in the analysis.
- Study of cross sections and longitudinal profile.
- Average gradient of the main riverbed (length and elevation of the beginning and end of the longer riverbed within each basin).
- Sinuosity ratio.
- Riverbed response to the proposed changes in it.
- Vegetation cover and land use.
- General overview of geology. Analysis of materials in beds and banks. In addition to particle size.
- Dragging capacity of sediments and scour analysis.
- Hydraulic conditions of the riverbed, flow and level of the maximum design flood for the return period used.
- History of flooding.

As for the design methodology, there is not one in particular defined to be used. Although all the necessary hydraulic studies for the design and location, both in plan and elevation of structures for pluvial water management should be performed and included in the calculation memory, according to a reasoned and documented approach. This must be performed from the respective desing flow for each structure, using the indicated return period in each case, and including a review of all existing structures to keep.

For the design of cross drainage works, should abide the provisions of the latest version of the circular notice HDS-5, named "Hydraulic Design of Highway Culverts". Free access document issued by the Federal Highway Administration (FHWA), Department of Transportation of the United States and the National Institute of Roads (NHI) and can be located on the website of said entity (**www.fhwa.dot.gov**).

The gradient of the structure to build must match that of the riverbed that contains the flow of water to pass under the road. Appropriate scour protection measures need to be taken for both the inlet and outlet of the line, and it might not have changes of direction. Inlet and outlet transitions (heads) should be appropriate according to the conditions of the site where the structure will be placed.

Also, cross drainage works shall consist of a single conduit and therefore drainage pipes laid in parallel are not allowed. Exceptions are made for very specific cases, depending on the importance of the road, but a prior consult will be needed as well as approval from the Supervisor Unit.

MINOR DRAINAGE

These structures should carry the flow to a receiving water body and in a way that it does not generate erosive phenomena. In case of steep gradients, the canalization must be done through energy-dissipation structures.

Some premises for the design of minor drainage, which may vary depending on the project to be carried out are:

- The minimum pipe diameter between manholes or heads, as well as paths under the road will be 80 cm. Among storm drains and wells, this may have 60 cm of diameter. For pluvial culvert systems where all water inlets are storm drains with grates or rods with 10 cm apart from each other; pipes with a 60 cm diameter and even among wells may be used, as long as their capacity allow it. Existing pipes in the area to intervene that do not comply with these diameters should be replaced.

- Closed conduits designed with a methodology only applicable to work on open canals, may have a maximum hydraulic brace of 0.75 times the pipe diameter.

- For the design of cross drainage works in general, it must meet the specifications set forth in the free access documents HDS-04 ("Introduction to Highway Hydraulics"), HDS-5 ("Hydraulic Design of

Highway Culverts”), HEC-22 (“Urban Drainage Manual”), all from the Federal Highway Administration (FHWA), Department of Transportation of the United States and the National Institute of Roads (NHI), available on the website **www.fhwa.dot.gov**. As a complement to the indicated therein, and to facilitate the calculation process, tools can be used in line with this methodology, such as the computational model “Culvert Master” and “HEC-RAS”. This includes every culverts of any cross type, whether circular, rectangular, and arched, among others.

- As far as possible, whenever various options of cross drainage works are feasible to use, the alternative less likely to become clogged with material dragged by water, must be selected. Preferably one culvert of larger diameter than several of smaller section but with the same hydraulic capacity.

- The minimum coating on the crown of the pipe to place must meet the specifications made by the manufacturer of the pipe, depending on the type of tube and type of bedding and filling proposed. To justify the minimum coating, the specification used must be attached, or at least a depth of 60 cm will be used in the case of Class III reinforced pipe that meets the C-76 standard. In cases where the above cannot be met, a reinforced concrete protection for the tube must be designed, demonstrating that it will have an adequate strength for the loads that is going to be submitted at that depth. If the pipe must be placed within 20 cm over the proposed slab protection, the designed must contemplate such situation, protecting the upper half of the pipe with a reinforced concrete structure properly designed.

- All low points of the road to be improved must have storm solution through appropriate structures, properly designed to vent site, which should be a course that is capable of receiving the discharge.

- The structures for pluvial water management (boxes, wells, drains, and headwalls) shall be constructed in reinforced concrete.

- When dimensioning works and electing their type, hydraulic and structural criteria must be considered. In general, for cross drainage works, culverts batteries will be accepted only in justified special cases, as it is intended that the hydraulic section of the structures have the least number of divisions, in order to avoid possible obstructions due to materials dragged by the analyzed riverbeds. Thus, for the passage of the given flow rate, culverts of larger diameter or dimensions must be used instead of several of smaller sections with the same capacity.

- At sites where the infrastructure of the road intercepts the runoff from natural terrain at the coronation of a cut, in berms and feet of

	batters or walls, as well as in places where it is deemed necessary, ditches of reinforced concrete with a minimum thickness of 10 cm will be designed. ■ The hydraulic section of reinforced concrete gutter should be designed according to the required capacity in regard to the flows that are required to be transported in each stretch. The gradient of the walls of each section of gutter to be used must comply with the applicable standards specified. ■ In areas where conditions of stability and bearing capacity of terrain permit, simple concrete gutters can be built as long as it is properly justified. ■ The minimum gradient in the longitudinal direction, of gutters and pipelines, will be 0.3%, except for gutters with flows with a design superior to 5 m^3/s, which must be thoroughly studied. ■ All surface runoff should be channeled so that it reaches a natural riverbed demonstrating that it has appropriate capacity. Infiltration wells or trenches may not be used. ■ Spillage of road surface water directly over filling embankments will not be allowed. These should be conducted using curbs, gutters or others, to the point of discharge properly designed with protection against erosion and, if necessary, with energy-dissipation structures for a proper management of velocities. ■ If at the project site there are minor drainage works, an inventory of all must be presented in order to know their location and hydraulic and structural characteristics, distinguishing between those that are capable of being exploited and the ones that should be replaced by others with higher hydraulic capacity. ■ For existing structures that are in good conditions and comply with the minimum diameters and other design premises, must be verified if its hydraulic capabilities are enough to evacuate the flow rate of the flood design, considering the appropriate safeguards in order to estimate if a replacement is necessary.
Considerations for works protection	All the necessary protection works to prevent erosion into riverbeds crossing the road o where rainwater of the project is discharged shall be designed, including the structures needed to vent. In addition, it is always necessary to calculate the water level and velocity at the outlet of cross drainage, to check if it is necessary to propose works to protect against erosion. The maximum allowable velocity to design at filled tube will be 5 m/s. The tractive force, whose minimum value is 0.1 kg /m2, defines the

minimum velocity.

When it is required to perform the vent of rainwater in the project area, it must comply with what is established in the General Law of Public Roads (Law No. 5060), Article 20: "*…All owners of real estate, by any means, are required to receive and let run inside their lands waters from the roads when so determined by the gradient and, when their estates are immediate to drainages of a road, these drainages must be clean, in perfect service and unobstructed…*".

In the case of building energy dissipaters, the outlet velocity of the water should be appropriate, in addition to its structural design and its foundation, so as to ensure that the field will have the appropriate structure for bearing capacity and there is no possibility of sliding, cracking or to suffering from any other damage along its lifespan. For the design of these structures, it is necessary to abide as described in the document HEC-14 ("Hydraulic Design of Energy Dissipaters for Culverts and Channels").

In the case of breakwaters, it should indicate and justify the minimum diameter of the rocks to place, the density of the rocks, the thickness of the protection, maximum gradient and the area duly leveled throughout where it should be placed. In addition, it should be indicated whether it is necessary to register with a mixture of mortar and indicate any necessary aspect for proper construction.

When it corresponds, what is indicated in the relevant documents from the Department of Transportation of the United States and the National Institute of Roads (NHI), available on the webpage **www.fhwa.dot.gov**, such as HEC-11 ("Design of Riprap Revetment") and HEC-20 ("Stream Stability at Highway Structures") should be used.

SCOUR

For the evaluation of the conditions of scour at each of the analyzed riverbeds, either by their own conditions, or by the placement of the proposed structures, the methodology of the Federal Highway Administration (FHWA), Department of Transportation the United States, may be used with the support of computer programs. When feasible, it is recommended to address what is indicated in the document HEC-18 ("Evaluating Scour at Bridges").

From the scour analysis, the scour profile of the superimposed riverbed or the original profile, will be developed (before estimated scour), including the location of the proposed structures (bastions, piers, foundations) in order to determine their effect and possible adjustment of the depth recommended for foundations, or otherwise, to change the position of the proposed structure.

	In structures where it is not considered necessary to perform this type of analysis, it shall be duly justified to the supervising unit.
Other considerations	The professional who will perform these studies must have at least 5 years of proven experience in various works related to this area and at least 10 works of hydrologic basin analysis of riverbeds with areas larger than 10 km^2, where hydrologic modeling or some kind of analysis was made using hydrographs and transfers of information between basins. In the area of hydraulics, they must have knowledge in the use of hydraulic models such as HEC-RAS or similar. This can vary depending on the type of project. If found any difference of opinion between what is stated in a foreign specification regarding to a national one which is in official use, shall be applied as established by the latter, as long as it exactly applies to the case study.

7.2 EL SALVADOR

Governing institution	Ministry of Public Works, Transportation, Housing and Urban Development (MOPTVDU)
Members of the Regional Technical Group (RTG)	■ Emilio Ventura. Climate Change Adaptation and Strategic Risk Management Office (DACGER). ■ Aníbal Henríquez. Planning Public Works Planning Office (DPOP).
Technical Reference Documents	■ *American Association of State Highway and Transportation Officials* (AASHTO) Highway Drainage Guidelines. ■ *Basic guidelines for climate change adaptation in the design of bridges in El Salvador*
Cartographic material	Topographic quadrants at a 1:50000 and 1:25000 scale, the altimeter sheets and aerial photographs are available at the Geographic and National Land Institute of National Registration Center. In the webpage, **mapas.snet.gob.sv** a map viewer of El Salvador can be found, provided by the geological service, the hydrologic service and meteorological service with useful reference information for the planning stages of road projects. If the event is located in San Salvador, the Planning Office of the Metropolitan Area of San Salvador (OPAMSS) has reference information.
Road data bases	The MOP has a database of road infrastructure of the country known as Road Management System of El Salvador (SIGESVIES) where it is possible to find types of culverts for minor drainage, Rolling type of the road, ditches, among others, and there is also a Bridge Management System (SAP).

Hydrologic and hydrometric instrumentation	The network implementation is managed by General Directorate of Environmental Monitoring (DGOA, formerly known as the National System of Territorial Studies, SNET) of Ministry of Environment and Natural Resources. The hydrologic monitoring network and the location of weather stations is available on the website: **www.snet.gob.sv**
Hydrologic information available	On the site, **www.snet.gob.sv** in the section "hydrology" it is possible to consult the location, type and condition of hydrometric or hydrographic stations in the country. In addition, there is available a flood history that dates back to 1921 in different parts of El Salvador. Both rainfall as hydrometric data must be purchased at the offices of the General Directorate of Environmental Monitoring of the Ministry of Environment and Natural Resources. It should be noted that the only station that has intensities recorded during the last 20 years, is the Ilopango Airport Station, so it contains representative measurements of the extraordinary events, tropical storms and hurricanes during that period.
Methodology for hydrologic analysis	**DELIMITATION AND CHARACTERISTICS OF THE BASIN** The drawings will provide the toponymical and curves of level enough to observe the correct path of divisions, these drawings should be presented in an appropriate scale of 1:25,000 or 1:10,000 or greater, as appropriate, in digital form where the delimitation of the different basins and/or sub-basins of the project are displayed, including the respective level curves. Simple schemes of basins and/or sub-basins delimitation are not accepted. From each basin it must be obtained the physical characteristics necessary for the calculation of flows generated therein, elaborating necessary summary charts where at least the following characteristics of each basin are specified: ■ Nomenclature ■ Drainage work planned (indicate dimensions, if there are existing drainage with parking). ■ Basin area from where the basin begins to the point of intersection with the road in km2, indicating in percentage the ratio of the area of influence of the basin with respect to the total surface thereof. ■ Length of the basin following the longest route possible of runoff. ■ Gradient between the head of the basin and pour pint (outlet). ■ Average resulting gradient. ■ Highway station that intercepts the basin ■ Different land uses, specifying their impact on the total basin

HYDROLOGIC DATA

It must take extra care when using Intensity – Duration – Frequency curves (IDF) of meteorological stations with no data of extreme events. Data of the rainfall intensities of extraordinary events recorded in the period from 1998 to 2012 should be incorporate and take into account, among which are: Hurricane Mitch, Stan and Agatha tropical storm, Tropical depression 12-E, but not limited to it.

Based on maximum daily rainfall data, it is recommended to do a graph of maximum rainfall frequency of the different months for each selected station.

For this process, data collected in selected rainfall stations can be used, generating the series of maximum rainfall in 24 hours, indicating year and month of occurrence, on which Gumbel distributions will be applied.

It is recommended to do a summary chart with selected stations and maximum rainfall adopted in it for different return periods (5, 10, 25, 50,100 and 500 years).

RUNOFF COEFFICIENT

The runoff coefficient of each basin is determined by its vegetation, type of crop and soil type, it must be specified the methodology used for this purpose or source of information. Such coefficient should consider the land use defined in the plan of territorial development of the area of influence.

PEAK FLOW CALCULATION

For the calculation of flow generated by the basins, proven methods must be followed.

To calculate maximum flows in natural basins, with an area of less than 1.5 km^2, the Rational Method can be used; while for larger areas, other methods available in specialized software will be applied for modeling hydrographic basins.

For the use of software for modeling hydrographic basins, a summary of the calculation procedure performed by the application should be included, showing inlet screens so that inlet and outlet data can be seen (software inlets - outlets) used for the calculation, with the respective description and analysis of the parameters used in the process.

Consideration due to variations in rainfall patterns in hydrologic analysis	Although the main cause is not the variation in rainfall patterns in the country, in the hydrologic analysis it should be considered an increase of 30% to 40% in rainfall intensities, when working with intensity-duration-frequency (IDF) curves for El Salvador, which have not been updated.
Methodology for hydraulic analysis	**RETURN PERIODS FOR DRAINAGE WORKS DESIGN** Gutters and longitudinal drainage works: 20 years Circular culverts <= 1.5 m : 25 years Box and circular culverts over 1.5 m : 50 years Bridges with lengths under 10 m : 100 years Bridges with lengths over 10 m: 200 years Scour estimated on bridges: 500 years **DESIGN METHODOLOGY** Among its activities shall be the following, but not necessarily limited to this: ■ Review hydraulic capacity and flow conditions of the existing drainage works. For the review of each existing work as well as for the design of each new work, the method of the FHWA must be used, with control of inlet and outlet reviewing the pipe capacity, water levels at the inlet and outlet and the riverbed velocity at the inlet and outlet. ■ Small existing works should be reviewed as culverts (culverts and box culverts), that is, must be calculated by control over inlet or outlet. A matrix with a list of all existing minor and major hydraulic works must be presented, and a consolidated inventory must be made, indicating the involvement of each of the listed works. Indicating whether they meet the requirements of hydraulic capacity, performance and its proposed intervention. **MINOR DRAINAGE** Physical and riverbed flow characteristics, flow rates to evacuate and hydraulic section that pipes should have, must be determined. As a general guideline, for pipes a minimum diameter of 91 cm (36”) will be used for maintainability; in addition, different alternatives for the materials to be used will be studied (concrete, metal, PVC, etc.), evaluating them with technical-economic criteria. Existing pipes with a minimum diameter under 91 cm (36”) will be kept, being in a good structural condition, not having exceeded their hydraulic capacity, and having a minimum coating (cushion). Existing pipes will be also evaluated to determine their current physical condition and its various inlet and outlet devices, also

determining its current capacity and future needs. If necessary, additions and/or modifications may be designed for each particular structure.

For surface drainage, it will seek to design a network or group of networks that allows the evacuation of surface runoff from the platform of the road and banks discharging into it, through a system of gutters (meaning any system of drainage structures) in free regime. For network, design will be considered the criteria that with respect to the type of elements and characteristics are defined by AASHTO drainage norm.

For underground drainage, it will propose elements of longitudinal drainage for underwater flows in cutting zones performed on steep gradients, and generally in any other area of the platform on which it is anticipated that underground runoff could affect the layers constituting the base or subbase of pavement structure. Likewise, a deep drainage could be necessary in cases that a longitudinal drainage is not sufficient and the cross one is not suitable.

In addition, sub drains will be performed where needed, following the current standards of AASHTO and FHWA. The use of geo-synthetics will be considered where technically and economically feasible.
Regarding to cross drainage, works needed to receive and download the water properly will be designed, by checking the hydraulic operating range of each, in order to determine if the existing section is sufficient for handling the calculation flow of the basin that serves. These requirements apply to both new construction and for the existing ones.

Wingwalls and headwalls, as well as coated riverbeds, will be designed with concrete, either plain or reinforced, as its dimension requires it. The use of stone masonry is limited to cases where there is availability and ease of stone use in the site.

MAJOR DRAINAGE

For the final design of Major Drainage structures, box culverts and arch culverts, the design parameters given in the specifications of regulations LRFD “Bridge Design Specifications” Edition 2004 of AASHTO, or later versions, must be used.

In the case of bridges, considering a safety factor for the hydraulic brace resulting from hydraulic-hydrologic analysis. To the resulting brace, a distance of 1.50 meters for mountainous regions, and 1.00 meter for plain areas should be added. The increase in the hydraulic brace is due to the excessive flow of water in rivers, transportation of debris, accumulations of clogged materials, etc.

Considerations for works protection	▪ Scour studies for bridges should be made and recommendations should be indicated for protection works upstream and downstream in the approximations, abutments and piles required for each drainage work. ▪ The length of the bridges must be equal or greater than the riverbed to avoid the problems of scour by contracting ▪ Design erosion control structures where erosion, either upstream or downstream of drain works exists. These erosion control structures can be coastal defenses, dissipating energy, dissipation pools, rock ramps, protecting bridge piers, etc. ▪ For the selection of the method of erosion control, the available materials in the study area or country, type of maintenance required, available labor, unit costs, etc., should be taken into account. Compare costs and technical feasibility of various methods of erosion control for the final selection of the material to be used. The erosion control methods can be, but are not limited to: riprap, gabions, geo-synthetics, confined cell sites (Geocells or equivalent), biotechnological techniques for erosion control. ▪ The study of river flow behavior should be considered in areas next to the possible location site of bridges, this in order to propose protection works for bridge abutments and approaches to flow impact. The protection should be from the sides of the abutments following on the banks of the river, a distance of at least 0.5 times the total length of the bridge, both upstream and downstream. Protection can be made of gabion walls, or any other system that provides protection conditions required by type and flow direction. ▪ For abutments and wingwalls, works of protection and drainage should be considered to channel the water coming from the roads to the bridge. ▪ Depending on the arrangement of the wingwalls with respect to the abutments, to the sides thereof may be allowed a protection zone 2 times the width of the bridge on either side thereof or 1.5 times the width of the bridge plus wingwalls of its abutments (whichever is greater), and to the river banks may be left a distance of at least 2 times the total length of the bridge to within the grounds.

7.3 GUATEMALA

Governing institution	Ministry of Communications, Infrastructure and Housing
Members of the Regional Technical Group (RTG)	■ Juan Carlos Galindo. ■ Dionisio Villegas Cansinos. ■ Víctor Vinicio Barrios. Roads General Directorate (DGC).
Technical Reference Documents	■ AASHTO (American Association of State Highway and Transportation Official). Drainage Manual. ■ Manual of Highways of Hydrology, Hydraulics and Drainage of Peru. ■ Drain Manual for Roads of Colombia. ■ Rural Roads Engineering. Field Guide to the best management practices of Rural Roads Management. ■ Recommendations for the planning and construction of underground drainages in road construction. ■ General Specifications for Construction of Roads and Bridges, 2001.
Cartographic material	The topographic sheets are in a scale of 1:50000, distributed by the National Geographic Institute "Engineer Alfredo Obiols Gómez", unit of the Ministry of Agriculture, Livestock and Nutrition. Website: **www.ign.gob.gt.** Maps of seismic hazards and landslides, Base maps of basins and rivers, geological map of Guatemala and other types of cartography can be found at the National Institute of Seismology, Volcanology, Meteorology and Hydrology (INSIVUMEH). Website: **www.insivumeh.gob.gt.** Also, maps of flood areas and landslides can be consulted at the National Coordinator for Disaster Reduction (CONRED). Website: **www.conred.gob.gt.** Finally, on the website of the Presidency Planning and Programming Secretariat (SEGEPLAN)_can be found the National System of Territorial Information (SINIT) which has a geo-portal that provides access to a set of resources and information related to space information of the country. The consultation website is **www.segeplan.gob.gt, section "On-line Systems"**/ National System of Territorial Information (SINIT).
Road data bases	DGC offices which have road data inventories are: ■ Traffic Engineering Department, DPE, from the Division of Planning and Studies. ■ Division Management Maintenance ■ The Performing Road Maintenance Unit (COVIAL), has also a road database.

Hydrologic and hydrometric instrumentation	Institutions that have instrumentation are the National Institute of Seismology, Volcanology, Meteorology and Hydrology (INSIVUMEH), the National Electrification Institute (INDE), the Ministry of Agriculture, Livestock and Nutrition (MAGA), the Municipal Water Company (EMPAGUA). In the website **www.insivumeh.gob.gt** the location of meteorological stations and the hydrometric network in the country from INSIVUMEH, can be consulted. As well as, hydrologic information of the country.
Hydrologic information available	Hydrologic information is obtained through a data application to the institutions mentioned in the previous section. Except for INDE which has an economic cost for the information. For INSIVUMEH, a public national institution has daily rainfall data, its coverage is between 10 and 86 years, depending on the meteorological station. Regarding series of maximum instant flows, between 10 and 40 years, depending on the hydrometric station. It is not uncommon to find incomplete time series due to externalities that have affected the data record.
Methodology for hydrologic analysis	The methodology used for hydrologic analysis is up to the consultants hired by DGC. As long as they justify their results adequately. In case of using the rational method, DGC limits their application to basins smaller than 1 km^2 (100 Ha). Some considerations for determining the design flow implemented in practice are: ■ The return period is a function of the lifespan of the road in the case of minor drainage design and for the design of major drainage structures. For bridges, the return period is 500 years. ■ Importance of the work. ■ The quality and reliability of the base information, as well as IDF curves. ■ Determine flow rates for events associated with different return periods, including the flow associated with the design return period.
Consideration due to variations in rainfall patterns in hydrologic analysis	Currently no consideration of changes in rainfall pattern is performed by the DGC. INSIVUMEH, the institution managing most of the hydro-meteorological data of the country, is performing a project to determine possible weather changes in the future.
Methodology for hydraulic analysis	The criteria used to differentiate major and minor drainages, is according to the criteria established by AASHTO. Minor drainage, , gutters and culverts to a diameter of 182.88 cm (72”). Major drainage: arch culverts and bridges. External consultants have questioned the border between the two rates because they do not have a formal policy or guidance as this manual.

<table>
<tr><td></td><td>RETURN PERIODS

Recommended frequency of storms values, referred to as return periods adopted by Guatemala in accordance with the rules applied and experience:
Major drainage: 500 years (DGC); 100 years (external consultants).
Minor drainage: 30 years.

The methodology generally used for hydraulic analysis on the DGC, follow these steps:
Establish design period.
Analysis of the drainage basin.
Calculation of rainfall intensity.
Flow calculation (Rational Method, synthetic unit hydrograph SCS, others).
Defining design flow.
Determining dimensions of drainage structure.
For structures minimum requirements, DGC has the General Specifications for Construction of Roads and Bridges, 2001 version. In Guatemala, it is known as the Blue Book of Roads.</td></tr>
<tr><td>Considerations for works protection</td><td>The need of additional or special protection is established based on basic studies of topography, hydrology, geology, geo-technical, hydraulics, detailed engineering for the road design. If required, the materials must comply with the specifications of the Blue Book of Roads.

In TDR, the considerations to protection works, if required, are indicated. These considerations are derived from pre-feasibility studies and feasibility studies, or previous assessments performed by DGC.

For protection works in river canals, the Technical Advisory of River Engineering from DGC is engaged in developing a manual for the treatment of these canals and specifications to add to the Blue Book of Roads.</td></tr>
</table>

7.4 HONDURAS

<table>
<tr><td>Governing institution</td><td>General Office of Public Works of the Infrastructure and Public Services Secretariat (INSEP).</td></tr>
<tr><td>Members of the Regional Technical Group (RTG)</td><td>Gustavo Ramón Suazo
Dénea Larissa Trejo
Hugo Fernando Martínez
Department of Hydraulic Works (INSEP)</td></tr>
</table>

Technical Reference Documents	▪ Manual of design and construction procedures of hydraulic works. (1998) Yoshihiro Takemoto. ▪ Manual of Roads of Hydrology, Hydraulics and Drainages of Peru. ▪ Drainages Manual for Roads of Colombia. ▪ Manual of Roads. VOLUME 6: Drains and Bridges (1996). Public Works, Transport and Housing Secretariat (SOPTRAVI).
Cartographic material	Topographic maps are available at the National Geographic Institute (IGN) in a scale of 1:50000 and 1:12500. There is also the portal of the Center of Disasters Information from the National Medical Library, in which can be found reference maps on flood risk, risk of landslides across the country. The website to access this information is: **http://cidbimena.desastres.hn/**, section "Additional resources".
Road data bases	Since Honduras lacks a mechanism of vial counting, the company or companies, individuals, legal persons and others interested in making road infrastructure projects will be forced to make their own traffic studies. And the results of these should be an integral part of the various studies to be performed. Also, it should contain a detailed explanation of the mechanisms that were used in these studies. Currently the Permanent Commission of Contingencies (COPECO) is creating a database of damage and location of bridges in the country, currently this work is still at a 38%, having collected the information in 7 of 18 departments.
Hydrologic and hydrometric instrumentation	Hydrologic and hydrometric instrumentation is managed by the Department of Hydrologic and Climatic Services, unit of the Hydric Resources Office from the Natural Resources Secretariat, The Hydrology Unit of the National Company of Electric Energy (ENEE) and the National Meteorological Service, unit of the Civil Aviation Office and the Permanent Contingency Commission (COPECO).
Hydrologic information available	In Honduras, within the observations network and hydro-meteorological measurements, it exists the measurement of rain by using 250 pluviometers located in different regions and about 45 pluviographs distributed according to the interests of different institutions, so it does not have a representative pattern of National Territory. In the south (Choluteca, Nacaome etc.) the density of the instruments is a lot higher compared to that in the east of the country.
Methodology for hydrologic analysis	Collection of at least 10 years of records from meteorological stations near the project area. Complete and detailed collection of data related to disasters, obtaining a relationship between them and the rainy conditions before the disaster. **RUNOFF COEFFICIENT** Values of C range from 0.05 for flat sandy areas to 0.95 for urban impermeable surfaces or clay soils. It is necessary to have a proper

	knowledge of the contribution surface to estimate acceptable values for C. The recommended values to use in Honduras are divided into those applicable to urban areas and those used in rural areas. The source from which these values are taken is according to consulted reference. **FLOW RATE CALCULATION** The rational formula is the method that has traditionally been used in the country for different designs of road hydraulic works. To keep the size of the maximum flows in the acceptable range, the extent of the area to apply the Rational Method is limited, ultimately to $4km^2$ (400 Ha), with surface areas up to 80% urbanized and concentration times of up to 5 minutes minimum. For larger areas, the use of unit hydrograph method is recommended, which is explained in the Roads, drainages and bridges Manual.
Consideration due to variations in rainfall patterns in hydrologic analysis	For the formulation of future projects, State of Honduras is currently developing a system that provides the information necessary taking into account the effects related to climate change in order to shield them against the challenges that such climatologic phenomenon presents. The estimation of flows derived from heavy rains is required, which generates extraordinary crescents in surface watercourses crossing the section of interest. The analysis is performed using deterministic methods sometimes ranging from empirical formulas to the hydrologic and hydraulic status of flood events, using nomograms, tables, even computer mathematical models. Other times it resorts to random variability of the hydrologic process and applies statistics and probabilistic formulations historical series. Due to the lack of specific research and collection and analysis of hydrometric information, design flows are often evaluated with indirect methods with foundations that rest on the rainfall-runoff relation, ranging from simple equations that relate a few physical parameters of the tributary surface of interest to complex mathematical simulations. Among these methods, we have the Rational Method and the Unit Hydrograph Method.
Methodology for hydraulic analysis	The magnitude of the variables involved in the different stages of the hydrologic cycle (rainfall, infiltration and runoff) and those that present physiography, directly affect the magnitude of the flow of the courses that concentrate the water which determine the size of the structure to be designed. It can be classified as follows: ■ Bridges on main roads (T_r: 50-100 years). ■ Bridges on secondary roads or culverts on main roads (T_r: 25 years). ■ Culvert on secondary roads, sewage culverts or roadside drainage (T_r: 5-10 years). ■ Stormd rains, curbs, pipelines (T_r: 1-2 years). It is noted that the concept of cross drainage is privileged compared to

lengthwise or of conductions; also major works (bridges) are favored over smaller (culverts). There are different methods to set the recurrence of design, i.e., average period in years when a flood equal or greater than the considered occurs. It is important to clarify the statistical significance of the term "average", this is that in a sufficiently long period an amount of flood as the one considered will occur, such that dividing the total period by said amount results a number of years representing the return period T_r. Often the recommended T_r value depends on the importance of each work and the cost/damage involved in the failure.

For the design of minor and major drains, in the Manual of Roads of Honduras, it is detailed presented the appropriate procedure that engineers must conduct.

Among the different considerations and recommendations, can be mentioned:

MAJOR DRAINAGE

The location of bridges is regulated by road altimetry and structural configuration (hydraulic considerations of placement where it should be evaluated the factors related to stability, the riverbed and the construction of future works), as well as its implementation with regard to watercourses (alignment with respect to the river, either oblique or perpendicular).

The length of the bridges may vary from one meter in the case of small bridges and culverts, and up to a few kilometers in the case of crossings over lakes or sea inlets. Also the support locations are conditioned by the following factors:

- Characteristic of the crossing over creek or river: that these do not interfere with the free flow of the flow rate should also be considered the presence of drag material in the river, which could damage the intermediate supports.

- Characteristics of foundation soil; should have such conditions that bear the pressures resulting from the actions of the structures, for which must have a capacity resistant to load and proper strain capacity.

- Type of material to be used in the structure; one of the most commonly used materials is the cyclopean concrete, reinforced concrete (which is a solid slab, which thicknesses can be up to 70 cm for spans up to 10 meters. Using ribbed slabs can be reached up to 25 meters in simply supported sections and 40 meters in hyper static sections) and pre-stressed concrete (it allows to economically reach up to 70 meters in simply supported beams and up to 200 meters in hyper static elements). In addition to concrete, steel is also used, but

<table>
<tr><td></td><td>it must be noted the high investment cost, due to the lack of metallurgical industry in the country, so when using these items, it must be imported, it also means higher maintenance cost.

■ Hydraulic considerations free height and width; leave a clearance from the lower level of the deck to the water level, to consider floating objects that the course may carry. Often designed under the influence of other nearby bridges.

MINOR DRAINAGE

■ The construction of roadside drainages to prevent water runoff in the area next to the shoulders, a minimum depth of 0.3 meters is recommended.

■ The built canals have trapezoidal section, usually of 0.6 to 0.8 meters of bottom width and 0.4 to 0.6 meters in depth.

■ As for culverts, there are two types, circular and rectangular, the first can be concrete or corrugated metal, and the latter supports significant coverages in large diameters making them ideal for use under high landfills in watercourses with significant volumes. The opposite occurs with concrete pipes that are usually prefabricated for diameters up to 1.6 m, which limit their use to small flow rates. Regarding rectangular culverts, its use is recommended for high flow rates or where are restrictions on the dam height.</td></tr>
<tr><td>Protection works</td><td>The conditions of water flow in relation to its brace, surface velocity, riverbed shape and superficial characteristics of the soil, as well as the shape and size of the obstructions created at supports located within the river, depending on the dragging velocity, they can be affected by soil erosion.

In the Manual of Roads of Honduras, are explained the different existing methods for the protection of a work subject to the erosive action of a watercourse. The elements commonly used for gradient protection are:
■ Dumped stone protection.
■ Protection of stone laid by hand.
■ Stone injected with cement grout.
■ Gabions.
■ Concrete placed in bags.
■ Concrete slabs.

It is essential to place a filter under any type of protection unless the bank material meets the filter requirements regarding the protection.

Some collection works such as catch basins and sinkholes of horizontal grid suffer systematically from clogging due to deposition of sediments. Similarly happens in small riverbeds with steep gradient with large</td></tr>
</table>

	volume dragging from the erosion of banks and fallen trees in the riverbed. Therefore it is recommended the use of precarious works for its reduced cost, which can be rapidly replaced in case of failure. Specific applications include temporary control dams, breakwaters perpendicular to the flow are mostly used (are built of rock, gabions, concrete or wood in some cases) and sediment curtains, either straw bales or geotextile membrane protection.

7.5 NICARAGUA

Governing institution	Ministry of Transport and Infrastructure (MTI)
Members of the Regional Technical Group (RTG)	■ Jerónimo Ignacio Sánchez. ■ Fidel Rodríguez Orozco. ■ Joaquin Guevara Arce. Office of Technical Studies of the Department of Planning and Management of Road Maintenance.
Technical Reference Documents	■ General specifications for the building of roads and bridges (NIC 2000). ■ *Hydraulic Guide for Design of Drainage Works and Rural Roads.* Edition 2004 and 2011. ■ Manual of hydro-technical studies review (2008). ■ *American Association of State Highway and Transportation Officials* (AASHTO) Highway Drainage Guidelines.
Cartographic material	The topographical maps, scale 1: 50000 are available at the National Institute of Territorial Studies (INETER) including aerial photographs at 1: 30.000. Some of the topographic maps can be downloaded from the following site: **www.ineter.gob.ni**/, section maps. Also, at INETER can be purchased flood and land use maps. The MTI has an updated map of the road network in 300 critical points, which can be consulted.
Road data bases	They have a road database called NICASAP, here checkups of each major drainage crossing are found. In addition to historical records of road damage.
Hydrologic and hydrometric instrumentation	The instrumentation network is managed by INETER. The location of the hydrometric network, as well as meteorological stations can be consulted on the website **www.ineter.gob.ni** / section “monitoring”.

Hydrologic information available	In the *Hydraulic Guide for Design of Drainage Works and Rural Roads* of 2011 can be found the location (coordinates) and area of influence of the most representative meteorological stations Nicaragua and recommended by MTI for hydrologic studies. In addition, to the area of influence of each station, determined by Thiessen polygons. Also, it is possible to verify the amount and type of records by station. Most of these have records of 30 years average.
Methodology for hydrologic analysis	Hydraulic Guide for Design of Drainage Works and Rural Roads of 2011 contains a flowchart that guides the reader to choose the method used to calculate the design flow. The first method shown is the rational formula, recommended for a drainage surface not exceeding 3 km^2 (300 Ha). Although it should be noted that, according to the aforementioned technical document, and as hydrology is not an exact science, this formula can be applied to tributary surfaces up to 12 km^2 (1200 Ha) or much higher, as long as another method of comparing the results is used. To calculate the concentration time (Tc), two formulas are shown: ▪ First, the one from Kirpich-Ramser, depends on the length of the main course of the river to the point of interest, the level difference between water and the cross of the drainage structure, and the average gradient of the basin. ▪ The second formula is the one from Basso and collaborators of the Central American Hydro meteorological Project (CAHP), which varies with respect of the Kirpich formula in the proportionally factor, reducing the time of concentration by 48%. Regardless of the formula used, the minimum Tc for calculating the flow rate must not be less than 5 min. The intensities are calculated by IDF curves or adjustment equations with coefficients that have been determined by INETER and can be found in the *Hydraulic Guide for Design of Drainage Works and Rural Roads*. The other alternative for calculating the maximum flow, and recommended on the hydraulic guide, in case of surfaces of drainage up to 3 km^2 (300 Ha), is the method of the Natural Resources Conservation Services (NRCS) formerly known as USSCS. This last method has adapted values from Curve Number (CN) for Nicaragua. Also, due the presence of swamps in some sub-basins of the autonomous regions of the north and south, RAAN and RAAS, are included in the hydraulic guide a section with a methodology that considers the reduction of flow by damming effect produced by swamps. **RUNOFF COEFFICIENTS** The runoff coefficient is taken from reference (Chow, 1994) which

<table>
<tr><td></td><td>includes, in addition to the vegetation cover and topography, rainfall frequency.

Finally, the runoff coefficient to be used, is a weighted coefficient with a result recommended to be compared with those shown by other literature sources; for example Engineering Hydraulics, edited by Rouse, and prepared by Kirpich, (1950) whose chart can be found in the technical reference document.</td></tr>
<tr><td>Consideration due to variations in rainfall patterns in hydrologic analysis</td><td>Exists the contract N° ES-007-2015 of technical assistance, which consists on the development of adaptive capacity to climate change. This technical assistance is carried out through a donation agreement FND C32 (Nordic funds) and implemented by a consortium of 4 companies: IDOM, NGC, METEOSIM-CONDISA.

The contract includes the following five components:
1) Institutional strengthening, coordination, agents involvement
2) Projections of climatic variables at regional level and identifying impacts on road transportation.
3) Standards reviews, design manuals, policies and legal instruments for introducing climate change criteria in design tools and roads maintenance.
4) Pre-investment studies of 30 projects, which include works climate change adaptation.
5) Implementation of 3 pilot projects based on 30 projects mentioned in the previous section.

The results of the new provisions regarding standards, design manuals, legal policies and instruments are expected to be in 2016.</td></tr>
<tr><td>Methodology for hydraulic analysis</td><td>In the Hydraulic Guide for Design of Drainage Works and Rural Roads the different methodologies for the design of structures for rural roads and low volume of traffic are explained. In each subsection of the guide there is an there are the design criteria and methodology for dimensioning drainage works such as standard speed-bump, trapezoidal speed-bump, culverts, boxes, fords with pipes and bridges not obstructing the natural flow and bridges.

MAJOR AND MINOR DRAINAGE

The distinction between major and minor drainage depends on the tributary area of the structure. Being 3 km^2 (300 Ha) the value that marks the limit between one and another.

PREMISES FOR THE DESIGN OF DRAINAGE WORKS

RETURN PERIODS

Outsloped dips: 2 years.
Tertiary road culverts: 15 years.
Box culverts for basins under 3 km^2 (300 Ha): 15 years.</td></tr>
</table>

Box culverts for basins over 3 km^2 (300 Ha): 25 years.
Fords with pipes: 2 years.
Box culverts and bridges for tertiary roads: 50 years.

OUTSLOPED DIPS

The analysis of the outslolped dips, is as that of a canal with a maximum water height of 30 cm. The rate is evaluated through the formula of Manning and flow through the continuity formula.

CULVERTS

The considerations for the design of culverts are as follows:

- Place them at crossroads over natural drainages, paralell to the flow course, so as to reduce the length of the tube and the affected area to a minimum.
- Use single large diameter pipes or a concrete box, instead of several tubes of smaller diameter, to prevent potential blockages.
- In broader canals, it is recommended to use multiple tubes to maintain the natural flow distribution through the channel.
- For sites with limited height, use "flattened tube", vaulted or arc section tubes to maximize capacity while minimizing height.
- Place culverts aligned on the bottom and in the middle part of the natural course so that the installation does not affect the stream alignment of the channel and the elevation of the channel bottom.
- Determine the design flow with Rational Method.
- For the calculation of culverts, should be established a radius of maximum load height inlet water between the proposed diameter, values in the range 1.00< (HW/D) <1.20.
- Ensure that the water level at the edge of the wall of the head is at least 40 cm of free ground water level.
- For maintenance reasons, the minimum diameter of 91.44 cm (36 ¨) is recommended.
- For dimensioning, the graphical method (nomogram) is used based on the formula for inlet control, developed by the Federal Highway Administration of the United States (FHWA). This method is shown in Hydraulic Guide for Design of Drainage Works and Rural Roads.

	BOX CULVERTS ■ To determine the design flow, use the Rational Method or NRCS according to the criteria of the basin size. ■ The depth of the backwater (He) or depth of water at the inlet, is an important factor in the discharge capacity of the conduit, the He or maximum hydraulic load of the box to determine the flow rate that the structure can evacuate, can be considered as the level of water at the edge of the wall head; ensure at least 40 cm free between the water level and the ground water. It is also recommended plating the area covering the backwater with masonry, in order to prevent erosion. ■ The dimensioning is affected by means of nomograms developed by the FHWA for concrete boxes and masonry boxes. **FORDS WITH PIPES** ■ To determine the design flow use the method NRCS, since typically watercourses that drain the fords are medium basins that require a bridge or permanent transit boxes. ■ The maximum hydraulic load associated with the flow return period should reach a maximum of 30 cm above the slab of the ford. ■ The design procedure is the combination of the methodology for the design of culverts, with the application of the formula of broad crested weirs. The total flow is the sum of the individual flows calculated by each method. ■ Where possible, use larger diameter pipes or dimensions in the case of boxes, rather than more with smaller dimensions. ■ This method is shown in Hydraulic Guide for Design of Drainage Works and Rural Roads. **BRIDGES** ■ It is recommended the use of "standardized designs" for minor bridges. ■ Use the NRCS procedure for the calculation of design flow for medium basins that require bridges to cross its waters over roads. ■ Use spans sufficiently long or a structure of appropriate length to avoid contracting the natural flow course of the riverbed. ■ Allow a minimum free edge, generally at least 1.0 meters between the bottom of the bridge beams and high water level expected with floating debris. ■ For small bridges, the recommended free edge depends on the flow

	rate passing through the channel. Being of 60 cm for flows between 3 and 30 m^3/s, and 90 cm for flow between 30 and 300 m^3/s. ■ Also for the free edge, the standard by the U.S Bureau of Reclamation can be used. Its value is obtained by the formula $BL=0.552\ (2.5\ Y)^{1/2}$, where "Y" is the brace or water depth in meters. ■ For dimensioning bridges, in the Hydraulic Guide for Design of Drainage Works and Rural Roads two cases are addressed: bridges that do not obstruct the natural flow and bridge structures that constrain the course. **ROUGHNESS COEFFICIENTS** It is recommended to do the estimation of hydraulic roughness by the method of Cowan, which depends on the type of materials present in the riverbed, surface irregularity degree, the variation of the canal cross section, the relative effect of obstructions, the vegetation and meander degree.
Considerations for the protection of works	The source documents make reference to the following works of protection in case of fords with pipes: ■ Use a structure or a slab large enough to protect the "wetted perimeter" of the natural riverbed. Add protection above the expected high water level. Allows a certain free edge, typical between 30 cm and 50 cm in the elevation between the top of the reinforced bearing surface (slab) and the high water level expected. ■ Protect the entire structure with waterproof screens, riprap, gabions, concrete slabs, or other scour protection. ■ Build foundations on scour resistant material (bedrock or coarse riprap) or below the expected scour. In case of bridges: ■ Protect access bridges upstream and downstream using forward walls (wingwalls), riprap, gabions, vegetation or other gradient protection where needed. ■ The foundations of the bridge must be built on solid ground. Build on materials not susceptible to scour (ideally bedrock or coarse riprap) or below the maximum expected scour. The depth of scour can be estimated with the criteria set out in the document. ■ Prevent the foundation or riverbed scour through local placement of heavy riprap, gabions cages or concrete reinforcement.

7.6 PANAMA

Governing institution	Ministry of Public Works of Panama (MOP)
Members of the Regional Technical Group (RTG)	▪ Porfirio Rangel. ▪ Jean Michael Guelfi. Drainage section. National Office of Studies and Design.
Technical Reference Documents	▪ *Approval Manual for Urbanization Plans**. Ministry of Public Works. Executive Directorate of Studies and Design. Department of Plan Review (2003). ▪ *Regional analysis of Maximum Floods in Panama. Period 1971-2006*. Company: Empresa de Transmisión Eléctrica S.A (2008). ▪ *Manual of Technical specifications of the Ministry of Public Works* ▪ *Drainage Study of Panama City* (1972). ▪ *Pluvial system design of Colón city* (1981). *Although this manual was designed to regulate the design and construction of residential areas, has also been chosen to use the hydrologic and hydraulic criteria for the design of drains on roads.
Cartographic material	Topographic maps are available on the National Geographic Institute Tommy Guardia at a scale of 1:25000 of all the country, and urban maps at a scale 1:5000 and 1:12500. In addition, aerial photographs and updated Orth-imagery is available. The Internet address where you can view the available material is **http://ignpanama.anati.gob.pa/** In addition, the National Civil Protection System (SINAPROC) has flood and landslides risk maps that can serve as a reference when planning projects.
Road data bases	There is a proposal for the creation of a road database, but up to the date of issuing this document, its creation has not been concretized.
Hydrologic and hydrometric instrumentation	The instrumentation network is responsibility, by law, of the Company: Empresa de Transmisión Eléctrica S.A (ETESA) of Panama, as is reflected in the document "Regional analysis of maximum floods of Panama. Period 1971-2006". It has a network of about 300 stations. Most of these automatic. According to company sources, the medium-term plan is to transmit information by telemetry and open access to users who need it. The instrumentation network location is available at the website **www.hidromet.com.pa** in the section "station network". In addition to historical hydrologic data in the section "hydrology".
Hydrologic information available	ETESA manages the network and also develops products based on the analysis of data. These can be purchased through request to the company. Furthermore, in the Manual of *approval of plans* can be found the

	intensities formulas used by the MOP to determine maximum flows for the design of drainage structures. These formulas are from the study Drainage of *Panama City* (1972) for the Pacific coast, except for the Azuero Peninsula, and arise from the statistical analysis of rainfall data over a period of 57 years. The data was obtained from the meteorological stations Balboa Heights and Balboa Docks, adjacent to the city of Panama and rainfall station at the University of Panama. From the analysis of information, intensity-duration-frequency curves (IDF) were obtained for return periods of 2, 5, 10, 25, 30, 50 years. For the Atlantic coast, recommended formulas are presented in the document *Pluvial system design of Colón city* (1981) and its records come from the meteorological station Cristobal, near the city of Colón. The observations of the rainfall were over a period of 22 years (1957-1979). The IDF curves are made for return periods of 2, 5, 10, 25, 30 and 50 years. Moreover, ETESA during 2015 supported the graduation work of Alcely Lao y Antonio Pérez titled "*Generation of Intensity Duration Frequency Relations for basins in the Republic of Panama*", in which IDF curves were updated for 10 major basins in the country.
Methodology for hydrologic analysis	According to the Manual *of approval of plans*, the use of the rational formula for tributary areas with no more than 2.5 km^2 (250 Hectares) is established. There is no specific formula to be used for the calculation of Concentration Time (Tc). If the tributary area exceeds 2.5 km^2 (250 Hectares), other methods of analysis can be used for calculating the maximum flow; including the one shown in the report *Regional analysis of maximum floods in Panama*. This report is an update to a previous version made in 1986 by professionals of the Hydro meteorological Department of the Institute of Hydraulic Resources and Electrification of Panama (IRHE), oldest institution in charge of energy and hydro meteorological issues in Panama. The data used for this regionalization come from ETESA stations and Panama Canal Authority (ACP). **RUNOFF COEFFICIENTS** Runoff coefficients to be used for hydrologic analysis, according to *Manual of approval of plans*, are the following: 0.85 sub urban areas and rapidly growing. 0.90 -1 deforested urban areas. 1.0 completely paved areas. Whatever the case, the minimum value of the runoff coefficient should not be less than 0.85.
Consideration due to variations in	Until the date of issue of this document, the MOP does not include on its terms of reference any consideration due to variations in rainfall patterns

<table>
<tr><td>rainfall patterns in hydrologic analysis</td><td>in hydrologic analysis. But on the other hand, ETESA, manager of most of hydro meteorological data, is making the first climate modeling to estimate future scenarios. Up to October 2015, the process of validating 40 years, from 1969-2009, is in its course, so that then they can be proposed to the future.</td></tr>
<tr><td>Methodology for hydraulic analysis</td><td>It must be designed for the whole tributary area that affects the drainage structure.

MAJOR AND MINOR DRAINAGE

It is considered as minor drainage gutters and culverts and drainage pipes, while major drainage are box culverts and bridges, but there is no value or parameter to differentiate them, as diameter, flow rate or basin area.

PREMISES FOR THE DESIGN OF DRAINAGE STRUCTURES

RETURN PERIODS

The return period used depends on the type of structure planned:

■ Pluvial culverts, pluvial systems spillways, pluvial drainage trenches: 20 years.
■ In case of having a connection to an existing pluvial culvert, it must be capable to evacuate the amount equivalent to the worst rain in 10 years.
■ Piping, channel retaining walls and other permanent structures of the pluvial system: 10 years.
■ In case of bridges on watercourses: 100 years.
■ Channeling of rivers and streams and pluvial boxes: 50 years.

CALCULATION METHODOLOGY

In the Manual of Approval of Plans a formula for hydraulic design is not specified.

If using Manning's Formula in the design, it is recommended the following roughness values (n):

■ Canals:
0.012 canals of plastered machicolation.
0.015 canals of smooth machicolation without plastering.
0.020 canals of smooth machicolation and soil bottom, natural excavations of winding tracing.
0.025 riverbed of smooth ground with ground vegetation.
0.030 ground riverbed, mud with debris or irregular due erosion.
0.035 natural excavation, debris covers with vegetation.</td></tr>
</table>

- Pipe:

0.013 concrete pipes.

If the Basin formula is used in the design, is recommended to use the following values for (m):

0.06 vitrified clay pipes.
0.14 masonry cement, concrete pipes, concrete coated canals.
0.05 rough stone masonry.
0.50 ground channels in good condition.
0.05 ground channels with vegetation and rock.
0.38 canals excavated in rock.
0.75 natural watercourses with vegetation and rocks.

DRAINAGES

- The design of canals and open roadside drainages must include sufficient unload places, maximal every 150.0 m. Sections of paved trapezoidal ditches should be designed with not less than 30 cm base.
- Pipes

The ratio of d/D for pipes must not exceed the value of 0.80 drainage capacity.

According to the aforementioned Manual, pipes cannot be less to 0.6 m diameter, although this value can be increased in the Terms of Reference, according to the considerations of the project and the main aim is to avoid the use of tube batteries or boxes.

- **Box culverts**

The ratio h/H should not exceed the value of 0.80.

In rivers and streams in areas of public use, boxing will not be allowed. Only open channeling is allowed.

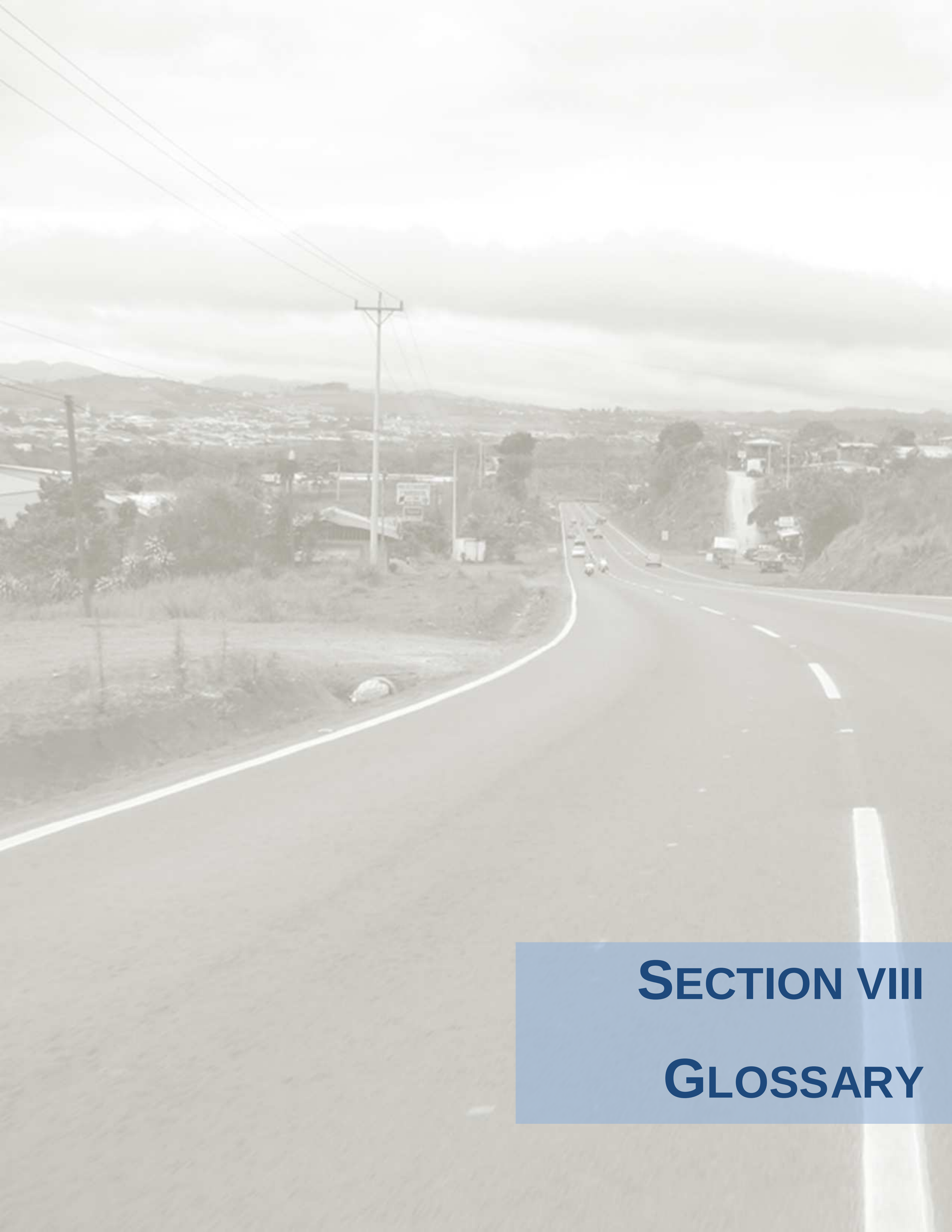
The hydraulic calculation contemplates that the structure is designed wider than vertical height and that it does not exceed the existing levels on land and basin in the periphery.

- **Vehicular bridges**

Perform a special topography and cross sections each 20 m to a distance 100 meters upstream and 100 meters downstream.

The clear distance between the High Water Level (NAME) and the lower level of beam, known as gauge, shall not be less than 1.80 m.

Considerations for protection works	■ Range of permissible average velocities to prevent erosion or sedimentation in the drainage structures, according to *Manual of Approval of Plans* : Reinforced concrete pipe (HR): 0.91 m/s (3 ft. /s) < v < 3.66 m/s (12 ft. /s). Masonry canals : 3.048 m/s (10 ft./s) < v Concrete canals: 4.57 m/s (15 ft./s) < v Boulder, sand and earth: v < 1.52 m/s (5 pies/s). In general, it is specified that the allowable average velocity must be within the following ranges: 1.0 m/s < v < 5 m/s. ■ The coating of H.R. pipes over the Crown should be 45 cm to the bottom of the pavement structure. ■ Any section of a designed canal, depending on the material that is going to be built, must maintain a gradient with an inclination that ensures stability and permanence of it. ■ In the discharge sites, headwalls should be constructed in order to hold the ends of the tubes and prevent erosion of the riverbeds and adjacent fillings. If deemed appropriate, it may be required to build stone pavement. If a discharge is performed to an existing system, the designer must verify the hydraulic capacity of the receiving system, and if it does not have the necessary capacity, the designer shall provide alternative solutions. The discharge can be done through pipes, ditches or paved channels in favor of the flow.
Other considerations	In designs and works, it is not allowed the inclusion of steel pipes. All pipes cross to the road to be built, must obligatory be of reinforced concrete.



SECTION VIII

GLOSSARY

GLOSSARY OF TERMS

Abutment. Part of the substructure, which supports the end of a whole section in a bridge. It also serves as a retaining wall for the fill in the back.

Aggradation. Aggradation is the accumulation of sediments in rivers and streams. Aggradation occurs when river sediments exceed the amount that such river can drag to its riverbed.

Alluvial soils. Soils developed from transportation and relatively deposited material (alluvial) characterized by a weak modification (or none) of the original material by the formation of soils.

Average velocity. Average velocity of a stream flowing in a conduit or canal in a given cross section or stretch. It is equal to the discharge divided by the area of the cross section of the stretch.

Backwater. Action or effect on a body of water in which the flow is slow or opposite to the normal flow of the riverbed. Areas of influence of obstructions in the riverbeds where water stops or its flow course is reversed, such as a narrow bridge, constructions or material of filling enclosing the area through which water must flow. In estuaries, this effect occurs because of the tides.

Base flow. Flow discharge derived from an underground source and as difference in a runoff. Sometimes considered to include flows from lakes or regulated reservoirs.

Slope. For engineering and architecture, the slope is the difference between the thickness of the lower section of the wall and the thickness of the upper sector, creating a gradient. This allows the wall to withstand the pressure of land behind it.

Brace. Also known as, flow depth or soaked is the depth of flow (usually represented with letter h) is the vertical distance from the lowest point of the canal section to the water surface.

Wingwall. Vertical wall on either side of a culvert.

Capacity of rain drainage installation. The maximum flow that can be transported or sewage by an installation of pluvial drainage without causing damage to public or private property.

Concentration time (tc). It is the travel time of a water particle from the hydraulically most remote point in the contribution area of the basin to the point under study.

Concrete. A concrete is a type of shallow foundations that can be used in reasonably homogeneous lands and with medium or high resistance to compression.

Contour lines. Line on a map that represents an outline or points of equal elevation.

Crescent. rapid elevation and usually short of the level of the waters of a riverbed up to a maximum, from which such level drops to lower velocity.

Critical flow. It is a theoretical condition in natural streams and represents the point of transition between subcritical and supercritical regimes. It corresponds to a Froude number equal to one.

Cross drainage. Allows the passage of water through natural riverbeds blocked by road infrastructure, so that no damage will occur in the last one. It comprises small and large works of passage, such as bridges and viaducts.

Cross section. A graph or plotter from the terrain elevations through the stream of the valley or portion of it, it is usually a line perpendicular to the stream or flow course.

Headwall. A wall built on top of a culvert to fix the adjacent soil.

Culvert. A closed conduit used for taking surface drainage of water under a road, railroad, canal or other impediment, it has from one to four cells or stretches which may be circular, rectangular or oval. The culvert has coated floor and also requires buttresses, heads, and aprons for its operation.

Curb. It is a prismatic, solid shape with a conditioned cross section. It is used to separate level or sloping surfaces, in order to visually delimit it confines a particular area or separates surfaces with different traffic types.

Cut. A portion of land surface or area from which land has been removed or will be removed by excavation.

Datum. Any surface level to which elevations are referred, usually using the Average Sea Level.

Deep foundation. They are based on the shear stress between the soil and the foundation to withstand the applied loads, or more accurately in the vertical friction between the foundation and the ground. They must be deeply located, to distribute over a large area, a large enough effort to support the load.

Degradation. General alteration of the ground surface due to denudation processes.

Design storm. A selected event, described in terms of probability of occurrence once in a given number of years for which it was designed and built or improved drainage flow controls.

Design storm. An estimate of the expected amount of rainfall in a given period of time.

Direct runoff. Part of the total runoff from rain that reaches the point of measurement within a short period after the storm began, and excludes the base cost.

Gutter. A ditch is a trench or canal that opens onto the sides of the roads of communication (roads, highways), due to its lower level, receives rainwater, and leads it to a place that does not cause damage or flooding.

Drain. A slotted, perforated, or buried pipe or other conduit (subsurface drain) or a trench (open drain) to carry the excesses of groundwater or surface water.

Drainage area. The drained area into a stream at a given point. It can be of different sizes by surface runoff, subsurface flow and base flow, but generally, the surface runoff area is considered as the drainage area.

Drainage work. A drainage work is a device used to make way for water, restoring the continuity of the course of riverbed mainly intercepted by linear works: roads, railways, etc.

Duration. The time period of a rain event.

Energy dissipater. A device used to reduce the energy of flowing water to prevent erosion.

Energy gradient. imaginary line showing the reduction or loss of the total load throughout a pipe or canal.

Environment. The total sum of all the external conditions that can act on a living organism or community to influence its development or existence.

Erosion. Erosion is the wearing or denudation of soils and rocks that produce different processes on the land surface.

Flood. A general term for all the detritus of material deposited or in transit through a stream including gravel, sand, silica, clay, and variations and mixtures thereof.

Flow rate peak. The maximum instantaneous flow of a rain condition in a specific location.

Flow rate. Normally the rate of water flow. A volume of fluid passing a point per unit time commonly expressed in cubic meters per second. The point, location, or structure where they are discharged into a body of natural water, waste water or drainage discharge pipe or open canal.

Foundation drainage. A tube or series of tubes that collect groundwater of the foundation or concrete structures to improve stability.

Free edge. vertical distance between the maximum water level, generated by a crescent design and the edge of a canal or the crest of the dam curtain or other hydraulic structure.

French drain. A drainage trench filled with gravel, material that transmits water; may contain a perforated tube.

Frequency distribution. In statistics, it is called a frequency distribution to data grouping into mutually exclusive categories indicating the number of observations in each category. This provides added value to the grouping of data. The frequency distribution shows the classified observations so that the number existing in each class can be viewed.

Gradient. Extent of deviation of a surface from the horizon, measured as a numerical ratio or percentage. Expressed as a ratio, the first number is usually the horizontal distance and the second is the vertical distance.

Hazard. Phenomenon, substance, human activity or dangerous condition that can result in death, injury or other health impacts, as well as property damage, the loss of livelihoods and services, social and economic disruption, or environmental damage. Hazard is determined by the intensity and frequency.

Hydraulic radius. It is the quotient between the area of the wet section and the wetted perimeter.

Hydraulics. It is the quotient between the area of the wet section and the wetted perimeter. That is, it studies the mechanical properties of the liquid depending on the forces to which they may be subjected.

Hydrograph. graph showing the variation with time of some hydrologic data observed, such as level outlet, velocity, sediment, etc.

Hydrographic basin. A hydrographic basin is an area drained by a single natural drainage system, i.e., that drains its waters into the sea through a single river, which flows to a single endorheic lake.

Hydrologic cycle: The hydrological cycle or water cycle is a biogeochemical cycle, in which there is a process of water circulation among the different parts of the hydrosphere, allowing water to pass from one physical state to another through chemical reactions.

Hydrology. (from the Greek *hydor*-, water) It is the scientific discipline devoted to the study of the waters of the Earth, including its presence, distribution and circulation through the hydrologic cycle and interactions with living things. It is also about chemical and physical properties of water at all stages.

Hyetograph. graph showing the intensity of rainfall versus time.

Infiltration Capacity. In hydrology, infiltration capacity is called the maximum velocity at which water penetrates soil. Infiltration capacity depends on many factors; unbundled and permeable soil will have a greater infiltration capacity than a clayey and compact soil.

Infiltration rate. The rate, usually expressed as inches per hour at which water moves through the soil profile.

Infiltration. Infiltration is the process by which water on the land surface enters the soil.

Laminar flow. Low velocity flow in which flow particles slide smoothly throughout straight parallel lines anywhere to the axis of a canal or tube.

Landfill. In civil engineering, it is called a landfill, the soil with which a land is filled to raise their level and form a plan of appropriate support for a work.

Longitudinal drainage. They are drainage trenches which are longitudinally arranged to the road or element to be protected, upstream thereof, in order to intercept water flows into these.

Lifespan. The period of time for which an installation or structure is expected to perform its function.

Loam. Loam is a type of sedimentary rock composed mainly of calcite and clay, usually predominant of calcite, giving it a whitish color with tones that can vary considerably according to the different proportions and compositions of the main minerals.

Natural drainage. The flow patterns of rainwater that runs over land on its pre state of development.

Nomogram. A nomogram or abacus is a graphical calculating device, a two-dimensional diagram that allows graphical and approximate calculation of a function of any number of variables. In its broadest conception, the nomograph simultaneously represents the set of equations that define a particular problem and the full range of solutions.

Open drain. A natural watercourse or built as an open canal that carries drainage water.

Permeability (soil). The quality of soil that allows water or air to move through it. Usually expressed in millimeters per hour.

Phreatic level. (1) The surface free of groundwater. (2) That area under atmospheric pressure underground, generally increasing rainfall with stations or other conditions.

Piers. They are the intermediate supports of bridges from two or more stretches. They must withstand the permanent loads and overloads without seats, be insensitive to the action of natural agents.

Pile. It is called pile a constructive element used for foundation works, which allows moving loads up to a tough layer of soil, when this is such a depth that makes it unfeasible, technically or economically, a more conventional foundation through concrete or plates.

Piled. Rigid liner cyclopean concrete (stone and mortar) used to protect from erosion, batters or the river.

Pit. An artificial open drainage outlet in which it drains the excess of surface water or underground rainfall or flow rates of a crescent, it can continuously or intermittently flow.

Porosity. The volume of pore space in soil or rock.

Probabilistic Analysis: procedure used to interpret a past record of hydrologic events, in terms of future probabilities of occurrence.

Project. Memory design and set of studies and construction plans, defining the works to be built in compliance with existing standards for each component of the project, as well as technical specifications and base budget for public bidding.

Retention time. The interval between the center of mass of rainfall and peak flow of the resulting runoff.

Risk management. Risk management is a structured approach for managing uncertainty related to a hazard, through a sequence of human activities that influence risk assessment, development strategies to manage and mitigate risk using management resources.

Risk. Risk is defined as the combination of the probability of an event occurring and its negative consequences. Factors that make it are hazard and vulnerability.

River. A river is a continuous stream of water, and more or less abundant, which flows into another stream or the sea. Rivers are either perennial or intermittent. Perennial rivers have water all year; intermittent, only during rainy season.

Road. It is called road to the side of the street or highway intended for traffic of vehicles.

Rocky bed. The roughly solid rock is placed on or below the surface of the ground. It can be soft, medium or hard, or hard and having a smooth or regular surface.

Roughness coefficient. The roughness coefficient n is a parameter that determines the degree of resistance offered by the walls and bottom of the canal to the fluid flow.

Routing: Routing is a mathematical procedure to predict the change in size, velocity and wave form of a time-dependent flow (Road Hydrograph), at one or more points throughout a stream (riverbed or canal).

Runoff coefficient. The runoff coefficient or runoff to the relationship between the plate of rainfall on a surface and the plate of water superficially draining.

Runoff. Water derived from the rainfall within a tributary basin, flowing over the surface of the soil or collected in canals or conduits.

Runoff: part of rainfall that flows into the riverbed field (surface runoff) or into the soil (subsurface runoff or interflow).

Scour. The clear and digging action of water flow, especially erosion downstream erosion caused by water stream in filtration apart from mud and slime for the flow bed and exterior riverbank of a curved canal.

Sediment. Solid material in suspension while being transported, or has been moved from its original site by air, water, gravity, or ice and is deposited on the surface of the ground.

Sedimentation. The process by which soils are deposited, waste and other materials on the ground surface or bodies or watercourses.

Shallow foundation. They are those that rely on shallow surface layers of the ground, by this having sufficient load capacity or due to buildings of secondary importance and relatively light.

Slime. Slime is an incoherent clastic sediment transported in suspension by rivers and wind, deposited on the bed of watercourses or on land that has been flooded. To be classified as such, the diameter of slime particles ranges from 0.0039 mm to 0.0625 mm.

Soil. The unconsolidated mineral and organic material on the immediate surface of the ground that serves as a natural environment for the growth of plants in soil. See also Alluvial Soil, Clay, Cohesive Soil, Loam, Permeability (Ground), Sand, Soil Horizon, Soil Profile, Subsoil, Surface Soil, Vegetation Cover.

Specific weight. It is called specific weight to the ratio between weight and volume of a substance.

Subcritical or regular flow. It has a low relative velocity and its depth is relatively large, the potential energy prevails. It corresponds to a regime flatness. It corresponds to a Froude number lower than one.

Substructure of the bridge. The substructure consists of all the elements required to support the superstructure and the road-elevated passage.

Supercritical or rapid flow. Has a relatively high velocity and shallowness, kinetic energy prevails. Typical of riverbeds of great gradient or mountain rivers. It corresponds to a Froude number higher than one.

Superstructure of the bridge. It is the top of a bridge, linking and keeping distance between one or more clear waters. The superstructure consists of the board (slab) directly supports loads and armor.

Surface drainage. A trench with permeable filler, usually containing stones and perforated tube to intercept groundwater or infiltrations.

Surface drainage. Set of works for the collection of rainwater, its channeling and evacuation to natural riverbeds, sewage systems or the ground phreatic layer.

Surface waters. Surface water are those found on the ground surface. This is caused by the runoff generated from rainfall or upwelling of groundwater. They may appear in a strong-flowing form, as in the case of streams, rivers and creeks, or quiet if it is about lakes, reservoirs, ponds, lagoons, wetlands, estuaries, oceans and seas.

Topographic map. Graphic demonstration of the characteristics of an area of land and in which the contour lines and elevations are shown according to a defined datum.

Tributary. River that does not flow into the sea, but at a more important river, which joins in a place, called confluence.

Underground drainage. Its mission is to prevent the access of water to upper layers of the road, especially to the steady, so the ground phreatic level and possible existing underground aquifers and streams, must be controlled.

Uniform flow. A state of uniform flow occurs when the average velocity and the cross-sectional area remains constant in all sections of a stretch.

Unit hydrograph: The resulting hydrograph of 1mm of excess rain (or 1inch or 1cm) evenly distributed in space over an area for a given duration.

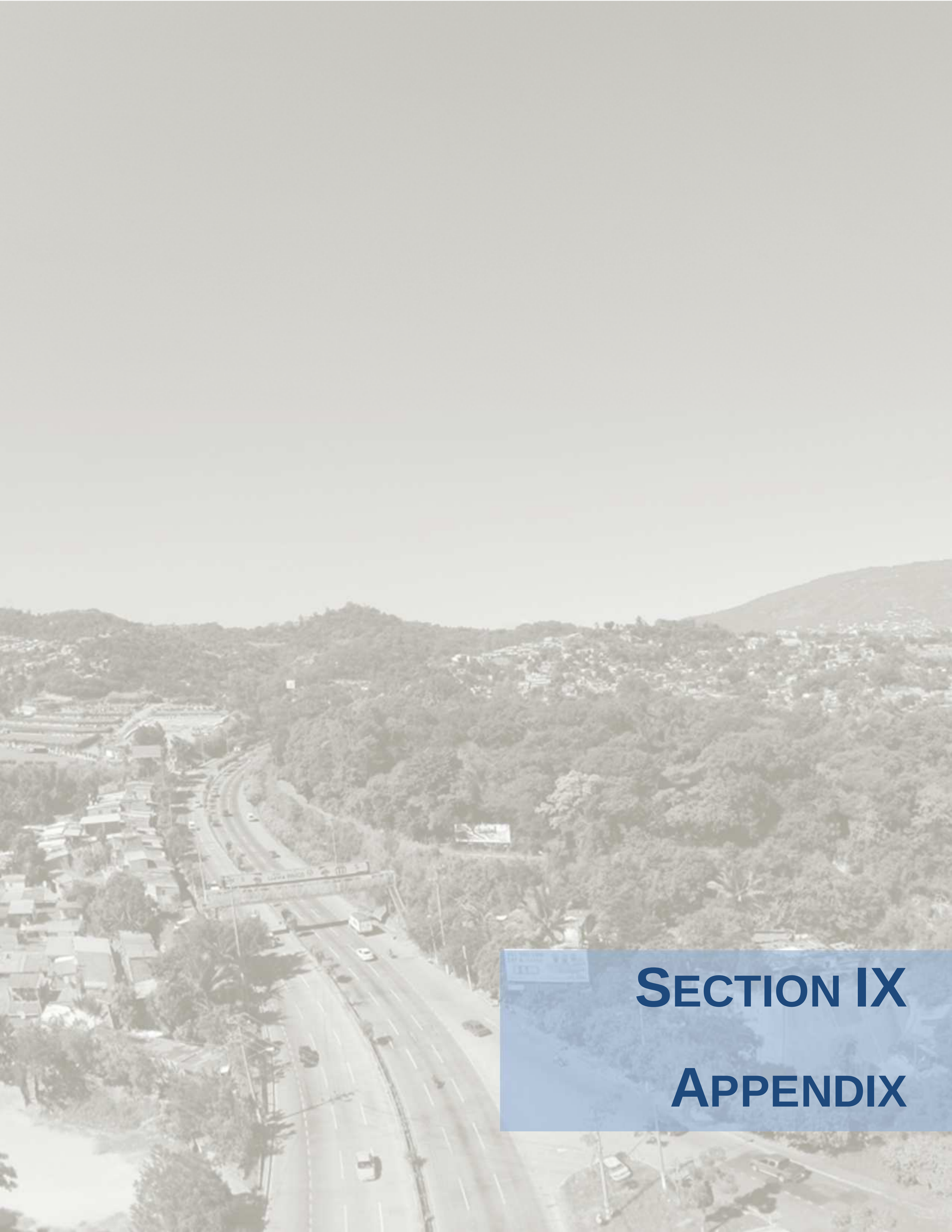
Vain. Space of a framed structure that is open between supports and beams.

Vulnerability. The characteristics and circumstances of a community, system or asset that make them susceptible to the damaging effects of a hazard.

Watercourse. Any river, stream, crique, creek, branch line, natural or artificial drainage or within rain runoff flowing continuously or intermittently.

Watershed. An imaginary line marks the hydrographic basin.

Wetted perimeter. It is the contour of the cross section, which is in contact with water.



SECTION IX

APPENDIX

Central American Hydrology and Hydraulics Manual for the design of drainage structures on roads

This form aims to collect initial information on hydrologic and hydraulic provisions in Central America for the design of drainage works on roads.
The answers will serve as inlet for implementing the regional diagnosis and respective manual

1. **Country**
   Country corresponding to the data provided
   Mark only one oval.
   - Costa Rica
   - El Salvador
   - Guatemala
   - Honduras
   - Nicaragua
   - Panama

2. **Name**
   Full name of the person who fills the form

   ____

3. **Institution where the person providing the information belongs**

   ____

4. **Unit or department within the institution**

   ____

5. **Occupation / Position**
   Occupation or position of the person providing the information

   ____

6. **Contact**
   Contact email of the person providing the information

REFERENCES FOR THE DESIGN OF DRAINAGE WORKS

7. **What unit or department within your organization, prepares the Terms of Reference (TOR) of bids for road projects?**

____
____
____
____
____

8. **Does the unit or department that prepares the TOR, have a reference document to establish the criteria for assessing the hydrologic and hydraulic parameters for the design of drainage works on roads?**

Mark only one oval.

- ⬭ Yes
- ⬭ No

9. **What is the name of the document (s) of reference used?**

Add the author and year of publication and version of the document

____
____
____
____
____

10. **If had a national reference document, do you know if it is based on some international standard? Which?**

____
____
____
____
____

11. **On what other institution, other than yours, do you depend for approval granting permits for the construction of road works?**

____

12.**In case of existing other institution involved in granting permits, in what aspects do they intervene?**

____

Considerations in hydrology, hydraulics, risk management and climate change adaptation when locating and planning road infrastructure

13. **Within your institution, is the scope of the activities to be performed defined according to the stage in which the road project?**

For example, activities taken place during the prefeasibility, feasibility or design stages

Mark only one oval.

- Yes
- No

14. **With what cartographic material does your office have?**

You can check more than one option

Check all that apply.

- ☐ Topographic maps
- ☐ Geological maps
- ☐ Land use maps
- ☐ Risk maps
- ☐ Water bodies
- ☐ Digital cartographic means
- ☐ Other: ______________________

15. **What is the scale of the maps you have?**

____

16. **Within your institution, do you have criteria to reduce the vulnerability of drainage structures?**

Mark only one oval.

- Yes
- No
- Other:

17. **Does your institution own a network of hydro meteorological instrumentation?**

E.g., Pluviographs, Pluviometer, Limn meters, Limn graphs, Scales

Mark only one oval.

- Yes
- No

18. **In case of having own instrumentation. What kind of instrumentation?**

Check all that apply.

- ☐ Pluviometers
- ☐ Pluviographs
- ☐ Limn meters
- ☐ Limn graphs
- ☐ Scales
- ☐ Other: ____________

19. **In case of NOT having a network of own instrumentation. Which institution is in charge of managing hydro meteorological information? How does your institution obtain it?**

____

20. **What type of data does your institution have for meteorological analysis?**

Check all that apply.

- ☐ Historical data (descriptive reports, watermarks)
- ☐ Instrumentation records
- ☐ Analyzed data (IDF curves, discharge curves)
- ☐ Other: ______________________

21. **Is there a database of road inventories within your institution? What unit or department has these road databases?**

Historical records of floods, historical records of damage in existing structures

____

22. **If the above answer is affirmative, is the historical information of rad damage included in this database?**

Mark only one oval.

- Yes
- No
- Other: ______________________

COLLECTION OF FIELD DATA

23. **Is there in your institution a form for the collection of data of road structures during field reconnaissance visits?**

Provide such form

Mark only one oval.

- Yes
- No

24. **Where does the information field visits turn? Who is in charge of managing the information?**

____
____
____
____
____

25. **Is any type of analysis done with the information collected in the field?**

Planning, risk management, maintenance plan

____
____
____
____
____

HYDROLOGIC AND HYDRAULIC CONSIDERATIONS

26. **Within your institution, is there a standardized methodology for the hydrologic analysis?**

Mark only one oval.

- Yes
- No

27. **If the above answer is NO, what is the methodology that is commonly used for hydrologic analysis?**

____

28. **Do you have criteria to include the variable of climate change on the hydrologic analysis?**

Mark only one oval.

- Yes
- No

29. **If the above answer is affirmative, what are the criteria?**

____
____
____
____
____

30. **Are there, within your institution, criteria for the design flow rate? Which?**

____

31.**Within your institution, is there a standardized methodology for hydraulic analysis according to the drainage structure?**

Mark only one oval.

- Yes
- No

32. **If not, what is the methodology that is commonly used for the analysis?**

____

33.**Is there in your office some software for hydrologic and hydraulic modeling or simulation?**

Mark only one oval.

- Yes
- No

34. **What software do you use?**

____

35. **Who uses the software?**

____

Protection Works

36. **As for the design of protection works carried out in your institution. Are there special considerations taken when implementing protection works in drainage structures?**

Mark only one oval.

- Yes
- No

37. **In what methodology is the design of protection structures based on?**

____

38. **ADDITIONAL INFORMATION**

This space is available for the respondent to add information deemed necessary to consider about the hydrologic and hydraulic criteria used in your institution for the design of drainage works on roads

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