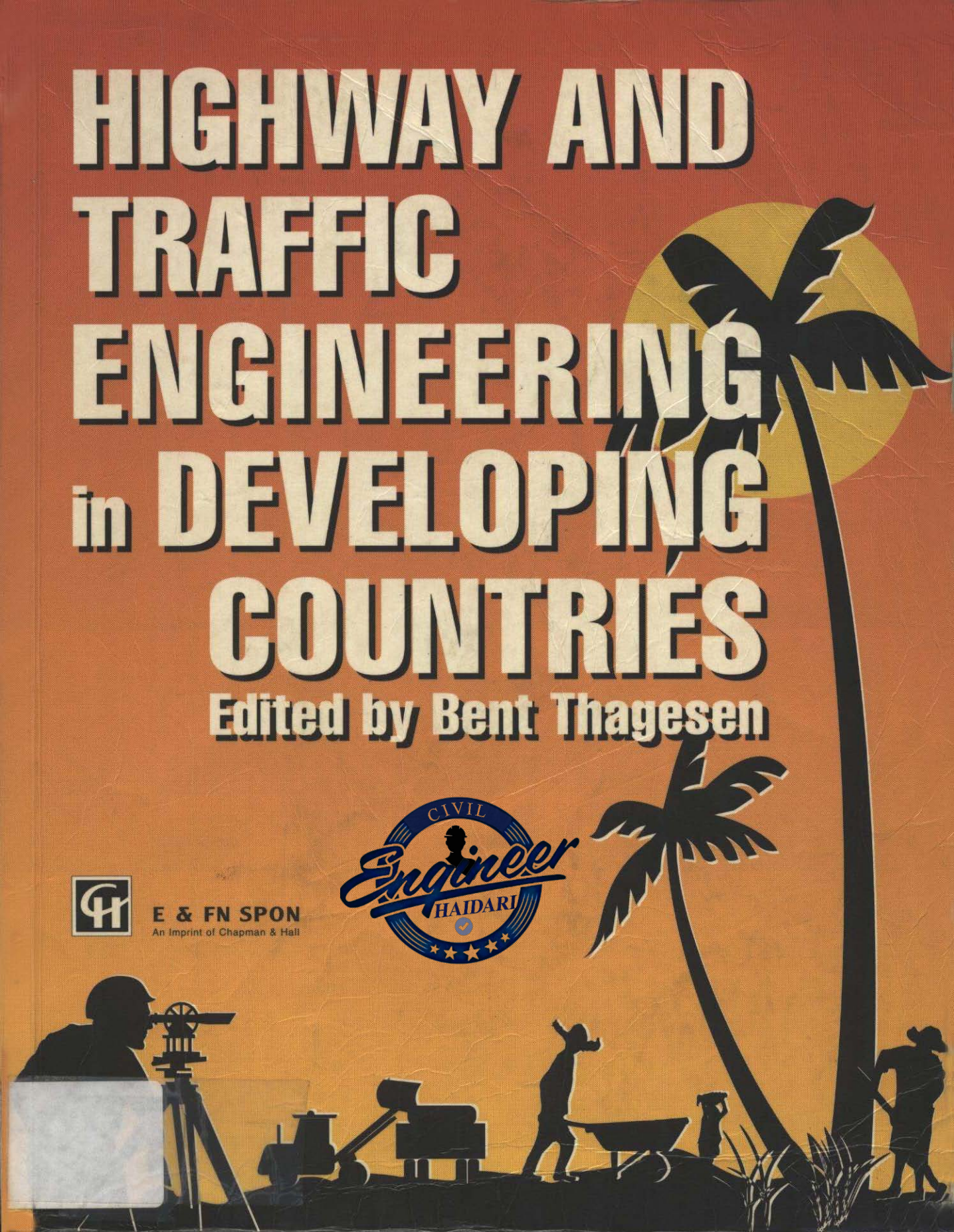
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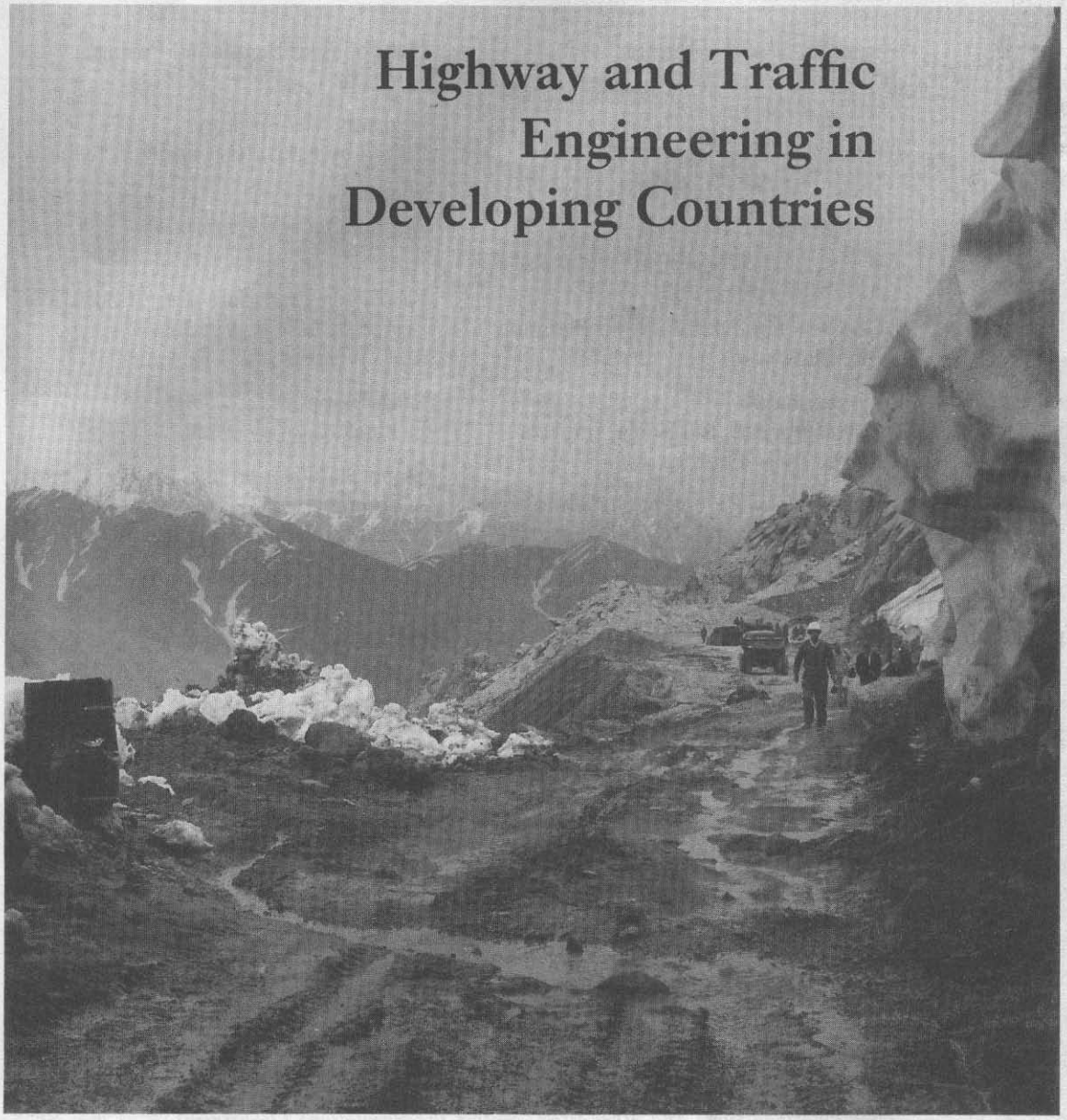


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Edited by

Bent Thagesen
Technical University of Denmark



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Preface

The main purpose of this book is to meet a pronounced need for a textbook on planning, design, construction, maintenance and management of roads and traffic in the traditional developing countries in Africa, Asia and Latin America. Most of these countries, which do not include the former Eastern Bloc countries, are situated in the tropics, where the natural conditions are different from related conditions in temperate regions. Also, the institutional issues and the financial problems confronting countries in the 'South' are usually different from the state of affairs in the 'North'. However, most existing textbooks on highway engineering are geographically biased and based on experience from industrialized countries with temperate climates, or they deal with specific problems, for instance, soil stabilization or road building in the tropics.

The aim of this book is to give a comprehensive account of the wide range of both technical and non-technical problems that may confront road engineers working in the Third World without giving a detailed coverage of methods and techniques. The book is designed primarily as a fundamental text for civil engineering students, but an additional objective is to offer a broader view of the subject for practising engineers.

The book does not purport to address the safety problems associated with testing of road materials and construction and maintenance of roads. Readers are expected to establish appropriate safety and health practices and determine applicability of national regulatory limitations prior to use of any method described in the book.

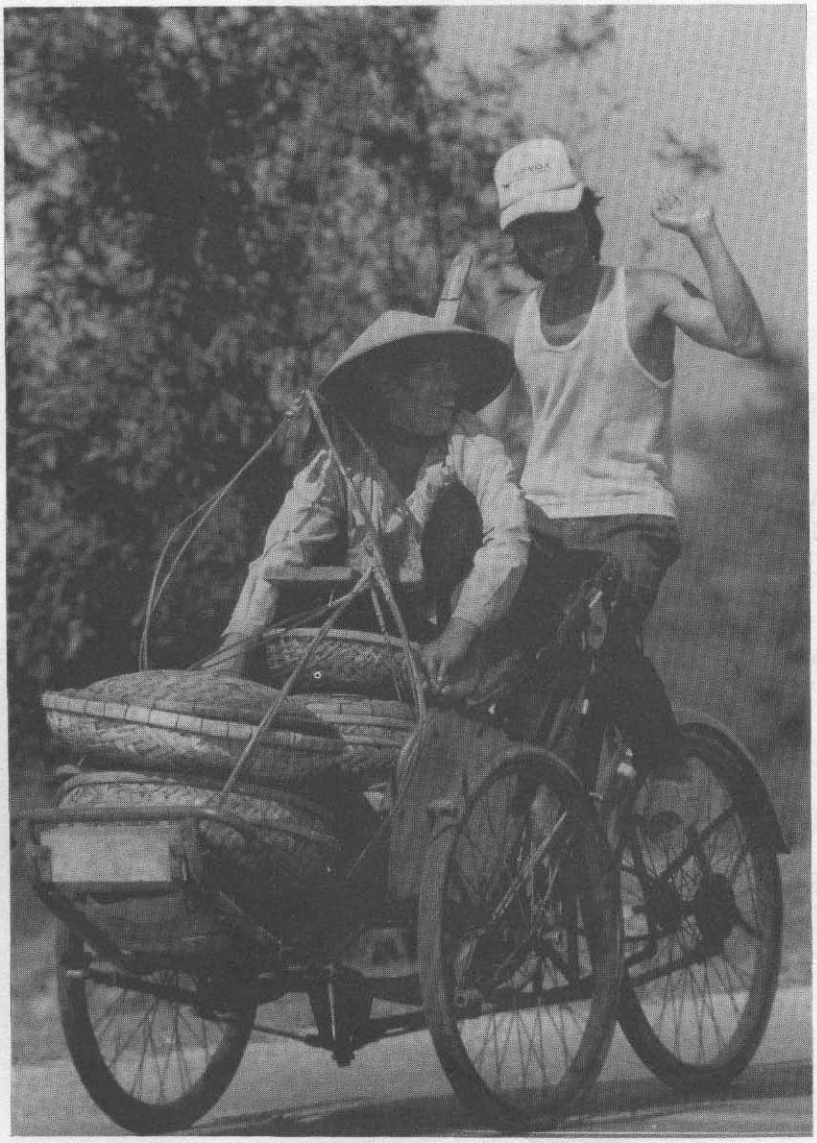
The text has been written with the assistance of a number of professionals with many years of experience gained in Africa, the Middle East, Asia and Central America. The names of the writers of the different chapters appear in the list of contributors, in the table of contents and under the headings of the chapters. I am indebted to them all for their contributions. My thanks also go to Wendy Taylor who helped with the preparation of Chapter 24; to Poul Harboe and Per Kirkemann who wrote background material for Chapter 25; to Arne Poulsen and Robin MacDonald who scrutinized various chapters; to Dr. Richard Robinson who assisted with manuscript review, and to Sanne Knudsen who did the proofreading.

Many of the illustrations have been reproduced from other publications. The sources are quoted below the illustrations and at the end of each chapter. The cover was designed by Ove Broo Sørensen. The preparation of the book has been financed partly by the COWI-fund and the Danish International Development Assistance (Danida). This help is gratefully acknowledged.

Bent Thagesen

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PART 1

Introduction



Chinese farmers on their way to the market. (Photo by Bent Thagesen) (Left) Happy cyclists in Vietnam. (Photo by Heine Pedersen)

Highways and development

1

Britha H. Mikkelsen, Cowiconsult.
Ole Møller, The Danish Transport Council.

1.1 INTRODUCTION

Justification for road works

A frequent justification for construction, rehabilitation and maintenance of highways in developing countries is that improved transport facilities are promoting development. This is most explicitly demonstrated in the great number of cases where road works are financed by grants from national or international aid agencies or by loans from the World Bank or one of the regional development banks.

However, development is an ambiguous concept. Development is defined in different ways by different scholars. Various theories differently emphasize factors which determine development and how a desired development can be promoted. Opinions on how different types of roads influence development have changed with time as more experience has been acquired.

It is found useful to introduce this book on highway and traffic engineering in developing countries with a discussion on development theory and the development effect of roads. Funds are extremely scarce in most developing countries and it is of paramount importance that investment in road works is thoroughly justified.

Theory

Development theory is generally interpreted to be theory about societies' change in the Third World. Development theory is systematic conceptualization of the conditions which determine the change of the societies.

Strategy

Based on a development theory it is possible to formulate a development strategy. A development strategy is a set of prescriptions for how to initiate and implement a development process.

Concept

A strategy must contain ideas about the direction of social change, i.e. about the goals which the strategy is intended to promote. Ideas about such goals is what constitutes the contents of the development concept.

For analytical purposes it is convenient to distinguish between development theory, development strategy and development concept. In reality there is often a close interrelationship between them. It is important to emphasize that different development theories are based on different development concepts. Common to all

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is that development is not a question of reaching one precisely defined state or stage.

GNP

The concept of 'development' has come to denote economic growth and material wealth. In day-to-day jargon development is associated with industrialization and signifies the process of change as well as a state of affairs. Translated into a simple measure 'development' is used almost synonymously with a high Gross National Product (GNP). International development organizations, the UN, OECD, the World Bank, etc., divide the world's economies according to the GNP measured per head, i.e. GNP per capita.

Low-income countries

Developing countries are summarily divided into: low-income economies, with 1992 GNP per person of US$675 or less; and middle-income economies, with 1992 GNP per person of $676–8355. A few developing countries belong in the group of high-income economies with GNP per capita of more than $8355. Table 1.1 presents summary data on GNP and population growth.

Economic growth

From Table 1.1 it appears that the poorest countries in Africa, the Middle East, and Latin America have a negative economic growth but a high population growth. The fact that a number of countries in East Asia, the so-called Newly Industrialized Countries (NIC), have been able to break away from the gloomy pattern by increasing their export earnings does not change the overall trend of a widening gap between rich and most poor countries.

Table 1.1 Summary data on GNP and population growth. All figures are weighted average (ref. 1).

Country group	*GNP per capita 1992 US$*	*Annual growth of GNP per capita 1980–92 %*	*Annual growth of population 1980–92 %*
South Asia	310	3.0	2.2
Sub-Saharan Africa	530	−0.8	3.0
East Asia & Pacific	760	6.1	1.6
Middle East & N. Africa	1950	−2.3	3.1
Europe & Central Asia	2080	–	1.0
Latin America & Caribbean	2690	−0.2	2.0
High-income economies	20570	2.3	0.6

Income gaps

The income gap between rich and poor countries is matched by big income gaps between social classes or between regions within many countries. The large income gap within individual countries is one of the characteristic features of under-development. For the same reason statistical averages of GNP per capita are problematic and often misleading. Further, the GNP tells us nothing about the benefit to the people of the economic activities contained in the measure.

Alternative measures

Critique of the crude economic development measure contained in the GNP has led to the development of alternative measures. The United Nations Development Programme (UNDP), a central body for the financing and co-ordination of technical assistance to developing countries, has tried to construct a composite index of human development that captures the three essential components of human life – longevity, knowledge and basic income for a decent living standard. The index is called the Human Development Index (HDI). HDI is a weighted

measure of life expectancy, literacy and income expressed in purchasing-power-adjusted international dollars.

There is no automatic link between GNP and human development. Countries at similar levels of GNP per capita may have vastly different human development indicators, depending on the use they have made of their national wealth (Table 1.2).

Table 1.2 Similar income, different HDI, 1991/92 (ref. 2).

Country	*GNP per capita US$*	*HDI rank*	*Life expectancy years*	*Adult literacy (%)*	*Infant mortality per 1000 live births*
GNP per capita around US$400–500					
Sri Lanka	500	90	71	89	24
Nicaragua	400	106	65	78	53
Pakistan	400	132	58	36	99
Guinea	500	173	44	27	135
GNP per capita around US$1000–1100					
Ecuador	1010	74	66	87	58
Jordan	1060	98	67	82	37
El Salvador	1090	112	65	75	46
Congo	1040	123	52	59	83
GNP per capita around US$2300–2600					
Chile	2360	38	72	94	17
Malaysia	2520	57	70	80	14
South Africa	2540	93	62	80	53
Iraq	2550	100	66	63	59

Western bias

The GNP concept and other statistical measures have been criticized for their interpretation of development in quantified terms. The critics maintain that the GNP measure has a Eurocentric or Western bias. It reflects the values of the industrialized societies where accumulation of capital is a driving motive behind transaction and people's communication. In industrialized societies it becomes 'natural' to measure development in money terms.

The human development index also has its limitations. It captures a few of people's choices and leaves out many that people may value highly – economic, social and political freedom, for example, and protection against violence, insecurity and discrimination to mention but a few.

Definition of development

Thus, it may be more appropriate to define development as 'sustainable improvement in the quality of life, especially for the poor and suppressed people'. Economic growth, social redistribution and political reforms may be the necessary means for obtaining genuine development in individual cases.

1.2 DEVELOPMENT THEORIES

Identification of the conditions under which societies can develop and give durable benefits to the people and identification of the causes of under-development and stagnation is the issue of development theories.

Post-war theories

Development theories are normally associated with the period after the Second World War. It is in the period of decolonization and the transition of the colonies to independent states that the theories of development and under-development start to flourish. Most of these theories carry some relationship to the classical economists or the Marxist tradition. However, in this latter period the development thinkers in many cases originate from the poor countries and from the colonies. Gandhi (India), Nkrumah (Ghana), for example, played a vital role in the de-colonization process in India and Africa. They differ from the classical theories in much more detailed analysis of the internal conditions of the poor societies in the perspective of their colonial background. In contrast to the classics the post-war development theories are more concerned about the development of the poor countries than of the rich. Therefore, development strategies for the poor countries become closely related to development theories.

Development aid

The post-colonial development debate is further centred on the role of development aid. It is particularly in the Western societies that aid continues to be a hot issue. Thus 'aid or trade', 'aid as obstacle', 'neo-imperialism', etc., are issues from the aid debate.

Essential questions

Essential questions that concern the post-war development theories are:

- What trends of development can be observed in the poor countries internally and in their relationship to the industrialized countries?
- What are the conditions under which the poor societies change in such a direction that most of the population benefit from this change?
- How can development, under-development or stagnation be explained?
- How can adequate development strategies be formulated with due consideration to economic, political and cultural realities?

Ideology

The ideological contents of the theories vary from a modernization perspective to solidarity with the wretched of the earth. Some focus on single explanatory factors, e.g. capital shortage, class struggle, population growth, education, the State, etc., others on comprehensive structural analysis of total social systems and historical explanations.

Modernization

The modernization theories belong in the category which perceive the world as divided into modern and traditional societies. Internally the poor countries are seen as consisting of a modern and a traditional sector that operates more or less independently. The concern of the modernization theories is first of all how to overcome traditional behaviour, values and attitudes and how traditional social structures can be transformed into modern states.

Representatives of this tradition are proponents of a development concept which equalizes development with such characteristics as social division of labour and high specialization, high productivity, self-sustained economic growth, an effective state bureaucracy and democracy. Development for the poor countries means change in the direction of these characteristics which describe the industrialized Western societies.

Dualism

This contrast between traditional and modern, in administration of the society, in people's way of thinking and behaving, is also embedded in the so-called theories of dualism. Well known among them is Rostow's theory of the stages of growth. It builds on a description of society as divided between a traditional economic sector and a modern sector. The traditional sector is characterized by

primitive technology, low productivity and production. The modern sector is characterized as dynamic, it uses advanced technology, productivity is high and production is directed towards the market.

Rostow's point is that all societies will have to pass through different stages before they reach full development comparable to that of the industrialized Western societies. When each of these stages will be reached by a given society depends on its natural and economic conditions. Political and cultural phenomena play a certain role but the main emphasis in Rostow's theory is on the economic development process.

Dependency

In stark opposition to the modernization theories a group of Latin American scholars laid the foundation for the most influential line of development thinking from the 1960s and onwards. Best known are the theories of dependency developed by A.G. Frank and Samir Amin.

A.G. Frank

A.G. Frank strongly rejected the idea that the developing countries consist of a modern and a traditional economic sector which function more or less independently of each other. His detailed studies of the Brazilian economy showed in contrast to the dualistic theories that a network of exploiting relationships directly connect the poorest peasants in Latin America with the directors of the transnational US companies. Frank describes these relationships as economic imperialism, which is the all-predominant cause of under-development and the primary hindrance for development. Indeed, rather than being precapitalist and traditional the tragedy of the poor economies is that they are an integral part of the capitalist world economy, according to Frank.

Samir Amin

There are many similarities between A.G. Frank's dependency theory and Samir Amin's theory of under-development. The latter has analysed the relations between Europe and Africa and finds similar patterns of exploitation as A.G. Frank did in studies of links between the US and Latin American economies.

The dependency theories had far-reaching implications for the anti-imperialist movement of the late 1960s and 1970s, and for the discussions of development strategies.

Global perspective

Historical developments in the post-war period of growing economic and political connections added a new dimension to development theories, e.g. the global perspective.

Brandt

The global perspective is predominant in the thinking which is laid down in the recommendations of the Brandt Commission reports (named after the former German Chancellor): *North–South: A Programme for Survival*, 1980; and *Common Crisis North–South: Co-operation for World Recovery*, 1983. The perspective in these reports is that the interdependence between the world economies requires that massive resources are transferred from the rich to the poor countries in order to revive or spur economic growth in these societies. Transfer of resources is not for the purpose of self-sacrifice. In the longer run it is supposed to be the only measure for survival of the economies of the rich world, who need external markets for their produce.

The perspectives of the Brandt reports have been refined in the works of the South Commission. *The Challenge to the South*, 1990, maintains the global interdependency perspective but as a new thing it shifts the analysis towards the internal weaknesses of financial mismanagement, institutional incapacity and weak human resources of the developing countries.

New division of labour

Other theories with a global perspective have dominated the debate of the 1980s. Thus, the theories of internationalization of capital broke with the stagnation perspective of the dependency theories. They were further elaborated in the discussions about a new international division of labour, propagated by several German researchers. Their studies focused on the mechanisms which could explain how a number of developing countries had embarked upon a genuine process of industrialization. These theories are generally based on Marxist thinking.

On the one hand a diminishing rate of return in the transnational companies of the industrialized countries, including Japan, triggered technological innovation which made possible deployment of whole production processes to the developing countries. These, on their side, were busy in establishing conditions that would attract international capital. Cheap infrastructure in Free Production Zones, for example, tax holidays for foreign investors, disciplined and cheap labour, often women workers, were used by the governments of the Newly Industrialized Countries (NICs) to attract foreign capital. These were a combination of 'push' and 'pull' factors.

Gunnar Myrdal

One of the prominent development thinkers during several decades in the post-war period, who in contrast to the above-mentioned scholars focused on the internal conditions of the developing countries, was Gunnar Myrdal. Myrdal broke with the one-factor-analysis which predominated the thinking of the neo-classical economists of the 1950s. Myrdal introduces six sets of variables which he finds are integral parts in the development process:

- production and income;
- conditions of production;
- standard of living;
- attitudes towards life and work;
- institutions;
- political interventions.

In many respects Myrdal was ahead of his time. It was remarkable at the time to introduce non-economic factors in the explanations of development and under-development, as Myrdal did.

Differentiation

The focus which increasingly has characterized the development theories of the 1970s and 1980s is the awareness of an ongoing differentiation between and within the Third World countries, i.e. the monolithic concept of the Third World is withering away. In this process the interest has also been shifting from the economic sphere to a greater concern for the cultural traditions and the socio-economic fabric of the developing countries and of the local societies within them.

1.3 DEVELOPMENT STRATEGIES

Development theories outline the conditions which must be fulfilled to initiate development processes. The strategies outline the prescriptions that are supposed to lead to certain goals.

Theories/ strategies

As mentioned the distinction between development theories and development strategies is not always sharp. Some theories are expressed more like strategies for how to change societies.

Implementation

Generally speaking, there are few comprehensive strategies which have been fully implemented in any one society. This should not be very surprising. We only need to turn to our own society to identify ambiguous and sometimes contradictory strategies being adhered to. It is also worthwhile to remember that development theories as well as development strategies rarely capture the full complexity of a given society. No society starts from point zero and rarely is the historical context the same from one society to another. One society is burdened by an extreme debt burden. Another by environmental degradation, population explosion and social deprivation. The development strategy must be adjusted accordingly. What often happens in concrete situations is that elements are adopted from complex strategies, or single-factor strategies are followed to solve specific problems.

Development strategies are the concern of planners, administrators, and not least of aid organizations, who need a comprehensive justification of resource allocations. Thus aid organizations have intensified their contribution to strategy formulation often to specific areas such as women in development, environmental protection, and human rights to mention a few. These partial strategies have all benefited from the theoretical development thinking about these same issues and their interplay with economic growth and social change.

Lacking consistency

It is tempting to think that there is a straightforward relationship between development theories and the development strategies followed by individual societies. However, only in rare cases is this the situation. Usually, 'real politics' is of a short-term nature. The consistency in the strategies is often difficult to identify at a given moment and less so over time. In the Western democracies the development strategy reflects a number of necessary compromises. In developing countries the policy may not be formulated within a democratic parliamentary system, and therefore the struggle between groups who are in favour of different development strategies is much more subtle.

Single-factor strategies

Western-oriented modernization theories predominated the scene until the late 1960s. The corresponding development strategies that were propagated had a similar touch but varied in their emphasis on single factors or agents of change. It was a widespread assumption that capital shortage was the primary barrier to development in the Third World. Capital investments was seen as a panacea to development.

Political scientists and sociologists each tended in their way to emphasize single factors. 'Nation building', 'formation of a bourgeoisie', 'promotion of the Protestant ethic', 'development of entrepreneurship', 'education', 'family planning' are examples of factors that were singled out – and to some degree still are – as being of particular centrality for spurring off development. Others have seen physical infrastructure – roads and telecommunication, water supply, etc., as the basis for further development and the justification for keeping up a state machinery.

Import substitution

In the 1960s the industrialization 'imperative' of the modernization theories strongly influenced the development strategies in the newly independent states. The strategy was to embark on import substitution of local production. But the technological capacity to manufacture the necessary machinery locally rarely existed, skilled manpower was not readily available, etc. Consequently, the import substitution industrialization started with capital-intensive production methods which did not require much skilled labour and for which machinery and management were imported from outside. A number of countries chose to follow this

strategy within the framework of a mixed economy. Kenya is an example of such a strategy where the private sector as well as the public sector and foreign as well as local capital were supported to play their important role in a capitalist development process.

This strategy led many countries into difficulties in the 1970s. Many industries were poorly managed and produced undercapacity. Large foreign debt burdens have accumulated since the incomes from agricultural or mineral exports on which most of the former colonies depended did not keep pace with the cost of imports.

As a result of government price controls and government-monopoly over marketing boards, peasants were poorly paid for their products. Local food production often stagnated, since the peasants had little motivation to produce, and the land resources were exhausted under high population pressure and inefficient production methods. Although the neglect of the agricultural sector had no direct connection with the import substitution strategy the results on national development have been serious.

Green revolution

This pattern seems to have characterized many Sub-Saharan African countries that have been the centre of strategy thinking in the last decade. In contrast many Asian countries embarked on 'the green revolution'. By applying scientific production methods, first to rice, later to wheat and other products, countries like the Philippines and India increased their grain production manifold within a short time. That these societies are still unable to feed all their people properly is a sign that their development strategies have other weaknesses, among which have been identified the problem of access to and distribution of resources.

Liberal strategies

Many countries that followed a development strategy biased towards industrialization, as well as countries which put more emphasis on the agricultural sector and 'the green revolution', ran into problems in the 1970s when oil prices escalated. Import prices for fertilizers, pesticides, machinery, etc., rose dramatically. In this situation some countries chose to redirect their development strategies. Since the purchasing power in their own countries was meagre, although the middle class had grown considerably, not least in Latin America, the idea of benefiting from scale of production and markets led the Newly Industrialized Countries to embark on a strategy of production for the world market. Central in this strategy was to offer cheap and disciplined labour, infrastructure and subsidies of many kinds to foreign investors. In this way they believed that they could offer employment to many of their people, earn the much needed foreign capital, and in the longer run manufacture the necessary technology themselves. In fact, the capacity to produce machinery and inputs to agriculture locally did increase substantially in the 1970s and 1980s. But generally, the benefits of the strategy were much more moderate than anticipated, leaving aside the social problems that often occurred due to abrupt cultural upheavals. Countries such as Malaysia and South Korea have proceeded along this road for about a decade while Thailand and the Philippines are newcomers in the trial of the liberal world market oriented development strategy.

Self-reliance strategies

The basic principle of Nyerere's African Socialism was 'self-reliance'. Its implication was to turn local production towards satisfaction of local needs and to reduce imports as much as possible. Since local resources in the form of organization, knowledge and technique were scarce, these had to be developed simultaneously. The emphasis laid on education of the people, i.e. a functional

literacy relevant to the local conditions of Tanzania, is an example of this strategy. So are the attempts to pull resources through 'collective self-reliance' as it was also practised for a few years in the East African Community between Tanzania, Kenya and Uganda.

In retrospect it can be concluded that the odds against the self-reliance strategy were too many. And this despite the fact that Tanzania received more foreign aid than other African countries. In fact, one has to conclude that the aid itself exacerbated Tanzania's problems in a number of cases, since neither the monetary nor human resources were available in Tanzania to make the donated development projects operative and sustainable.

Anti-imperialist strategies

The strategic implications of the dependency theories were also self-reliance or collective self-reliance, for example in the form of more South–South co-operation. The dependency theories came to play a vital role in the formulation of anti-imperialist strategies of the 1960s and 1970s in Vietnam and the Portuguese colonies, for example.

The dependency thinkers provoked the conventional Western-oriented modernization perspective of the 1960s. By reverting the theoretical perspective on imperialism and in changing the terminology to talk about under-development, they laid new grounds for development strategy formulation. The implications of the dependency theories for a relevant development strategy were that the poor countries had to break their ties with the rich countries. Only by severing the exploitative relationships and focusing attention on their internal needs would they be able to embark on genuine development.

The reverse picture of imperialism which they presented appealed strongly to the Third World populations, who were encouraged in their anti-colonial struggle. It confirmed the thinking, that the root causes of the 'late start' lay outside rather than within their societies. The experience of Angola and Mozambique shows how difficult it is to turn the energies of an anti-imperialist struggle into mobilization of internal resources for self-reliant development. The task was further complicated in the case of these two countries by the provocations and sabotage from their powerful neighbour South Africa to which they were subjected for decades. The future now bears hope for co-operation.

1.4 STRUCTURAL ADJUSTMENT

The formulation of development strategies during recent years has been strongly influenced by the World Bank and the International Monetary Fund (IMF).

World Bank and IMF

The World Bank is the most important international, financial institution providing loans for development projects. The IMF provides credits in order to improve balance of payment.

Success of liberal strategies

The relative success of the societies that have followed a liberal, world market oriented development strategy made the World Bank and IMF adopt many of the elements from this strategy in their advice to other societies. Not least the example of South Korea which is known to have introduced agricultural reforms in the wake of its industrialization 'miracle' has been taken as an example for other countries to follow.

Economic liberalization

The essence of the structural adjustment strategy which the World Bank and IMF has advocated during the 1980s is to liberalize the economies in order to bring about an economic recovery. The idea is to mobilize local resources more effectively than has been accomplished by often inefficient state bureaucracies. The recommended means are cutbacks in the public services, but also interventions in price policies in favour of the farmers in particular. Generally, price increases have been directed at the export cash crops and less at local foodstuffs.

Shift in emphasis

The balance of the structural adjustment policy has shifted the emphasis of the related development strategies from industry to agriculture, from social infrastructure, e.g. education and health, to the productive sectors, from the public to the private sector and from the local market to the world market.

The influence of the World Bank on the formulation of development strategies is not new. The Bank exerted strong influence on the formulation of the green revolution policy as it did on the strategies chosen by the 'industrializers' of the 1960s. This influence has been direct in the sense of the conditionalities applied by the Bank in its lending policy as well as through the placement of World Bank advisors in the Ministries of Economic Planning in a number of countries. The formulation of strategies has also been influenced by the returning Third World scholars who were educated in the neo-classical economic tradition in American and European institutions.

Debt problems

When the influence of the World Bank and IMF on the formulation of development strategies seems stronger today than it did 10–20 years ago it is because so many developing countries have run into very serious debt problems. Country after country has been forced to seek assistance from outside to ease their economic problems. Many have turned to the IMF for relief. With the soaring interest rates in the 1980s they have had to take on new loans just to repay the previous interest debts.

Political influence

From the borrower's side the problem of having to approach the IMF is much more than an economic one. The influence of the IMF has become just as much a political influence, as the discussions between a number of countries and the IMF have proved.

Unpleasant conditionalities

But not only socialist societies have experienced how their access to aid from the World Bank and IMF and even from bilateral aid organizations has been subject to their giving in to unpleasant conditionalities. These normally include stark cuts in public expenses, devaluation of the currency, liberalization of import restrictions and removal of subsidies – in particular on foodstuffs. The latter has led to repeated food riots in a number of Latin American cities, in Cairo, in Zambia and many other places, where the urban poor have seen the food prices double overnight.

Countries give in

Country after country is giving in to the conditionalities of the IMF, the World Bank and liberal donors. Most governments experience this as some kind of arm twisting. Others, like China, are voluntarily embarking on a development strategy with strong liberalistic traits.

Results uncertain

It still remains to be seen whether the competitive switch in the development strategies will be better able to bring about development than strategies promoting import substitution or self-reliance. The trial is not only whether wealth and prosperity can be accomplished for a minority of the population. The trial is much

more how to bring about development for the poor and suppressed people. In the short run the results of the current adjustment strategy may be impressive on the surface and satisfy those who identify development with material wealth for the privileged of the type we know in the Western world. The harsh reality of this strategy is a widening gap between rich and poor. In the long run a genuine development strategy will have to address the question of how to involve the poor and suppressed in the formulation of their own destiny.

Recent inputs

Recent inputs to development strategy thinking prove that this is an area of continuous change. It is significant that leading organizations like the World Bank, the UNDP and UNICEF (United Nations Children's Fund) focus on *Poverty, Human Development and Adjustment with a Human Face* in recent publications. The severity of the problems faced by scores of Third World countries, and the Sub-Saharan countries in particular, escalates year by year as their populations increase. There is reason to hope the advances in theoretical and strategical development thinking as witnessed in the mentioned reports and in writings of the South Commission and of numerous Third World scholars is an important step towards a better understanding of the problems and hence their solution.

1.5 HIGHWAYS AND THE DEVELOPMENT PROCESS

Theory

There is no well-established theory on the relationship between highways and development. It is generally accepted, however, that a certain minimum number of highways and other transport facilities is absolutely essential to allow and encourage economic and social development. Economic development is conditioned by a certain level of transport infrastructure. On the other hand it requires a certain degree of organization, know-how and economic development for a society to construct and maintain a transport infrastructure of a given level.

Debate

During the last three decades development planners have discussed how much emphasis should be put on transport investments compared to investments in other sectors. The professional debate has been heated between those who have seen transport as a leading sector capable of inducing economic development, and those who have seen transport investments as needing to respond only to actual traffic demands.

Trunk roads

Some important aspects have been generally agreed upon. In areas where there is already access for motor vehicles, road investments can generally not be expected to generate economic activity. Investments in improving major highways in developing countries will lead to reduced vehicle operating costs and travel time, but they will seldom induce economic development.

An exception is cases where an isolated region is connected to the rest of the country by a new trunk road. This will allow that region to be integrated in the economic and social life of the country by exchange of goods, information and people.

Rural access roads

Investments in rural access roads that are opening up isolated areas or reducing transport costs dramatically will often have a good chance of generating social and economic development, provided there is a development potential in the areas affected. The impact of rural access road investments will be discussed below.

1.6 EFFECTS OF RURAL ACCESS ROADS

Rural access roads are normally narrow earth and gravel roads of simple engineering standard. They form the last link in the road network connecting the rural areas to the primary roads, and they carry relatively little traffic.

Impact categories

Improvement of existing access roads or construction of new ones may affect the area of influence in many ways. Empirical evidence from a number of impact studies indicates the following impact categories:

- employment generation during the construction phase, especially where labour-intensive methods are applied; e.g. from Kenya there have been reported 2000–3000 man-days per km road constructed;
- improved access for rural dwellers to employment opportunities and health centres, hospitals, district offices, schools, etc. in nearby towns;
- increased social cohesiveness and national integration;
- improved supply and lower prices of imported consumer goods;
- agricultural development indicated by higher yields, changing land use, increasing use of modern farm inputs, and growing production for the market.

Negative impact

On the negative side it can be noted that improved road access to rural areas may lead to dissolution of local cultural values, ousting of local cottage industry by import of cheap manufactured goods, and accelerated deforestation and soil erosion caused by intensified agricultural production.

Agricultural development

Agricultural development is often the main objective of rural access road investments. The likely mechanisms of change in agriculture caused by improved road access can broadly be described as follows. Reduced transport costs will bring about higher prices of agricultural produce at the farm gate and lower costs of farm input. The farmers will often – but certainly not always – respond to that by producing more for the market. Moreover, improved access means that information on prices, market opportunities, new crops and farming technologies are passed on more readily.

Development conditions

It appears from various impact studies that the actual net value added in agriculture is often significantly less than what had been anticipated in the planning phase. To what extent and at which speed the anticipated agricultural development will materialize depends on the following factors:

- The development potential of the area given by natural conditions.
- The level of access constraint before road construction, and thus the magnitude of transport costs savings. Transport cost savings are to be significant (e.g. shift of transport mode) if agricultural production should be affected.
- The transport costs' relative proportion of the total price of agricultural produce. Transport cost savings will have a larger impact on 'heavy' produce with low price per ton and highly perishable produce.
- The degree to which transport cost savings are passed on to the farmers through lower transport rates. This depends on the competition among transporters and middlemen.
- The awareness and attitudes of the farmers to innovations and to changing conditions of production.
- Bottlenecks other than access, which are hampering agricultural development, e.g. lack of market, credit, extension service, farm input, etc.

According to the impact studies it is common that a combination of the above factors are curtailing the anticipated agricultural development.

Passenger traffic

A number of impact studies report that the increase in passenger trips on the improved roads often exceeds expectations significantly. This social benefit, however, is hard to quantify in monetary terms.

Altogether the demand for passenger transport seems to have a higher price elasticity than the demand for goods transport, price elasticity is the degree of sensitivity of transport demand to a change in transport price.

A later chapter includes a description of how to estimate development benefits of rural access roads in an economic appraisal.

1.7 HISTORICAL DEVELOPMENT

Road network

The road network of developing countries is more sparse, both measured per square km and per inhabitant, than that of developed countries. Table 1.3 gives the densities of the road networks of some selected countries.

In many Third World countries the most important inter-urban trunk roads linking the provincial capitals are in place, while the network of secondary and access roads in rural areas is far from fully developed.

Table 1.3 Road densities for selected countries 1988 (ref. 1).

Country	*Km paved roads per million persons*
Burkina Faso	21
Bangladesh	59
Ethiopia	84
Tanzania	156
Indonesia	160
Honduras	335
Zimbabwe	1 389
United Kingdom	6 174
Denmark	13 775
United States	14 172

Often the roads already in place are in a poor state of repair due to inadequate maintenance. For example a recent study of the road network of Tanzania, cf. ref. (3), estimated that more than 60% of the trunk road network is in an unsatisfactorily poor state of repair. In addition, the study points out that the condition of most rural access roads is very poor and transport to villages is both difficult and costly. A large number of rural access roads are not even passable by motor vehicles during the dry season.

The current extent and condition of the road network in developing countries is a result of the previous historical development and the present economic situation of these countries.

Colonial period

In most Third World countries the backbone of the existing transport infrastructure in terms of harbours, major railway lines and trunk roads was established during the colonial period. The purpose of this infrastructure was to evacuate raw materials from mining areas, plantations and agricultural estates to the coast for shipment to Europe.

Thus, most of the new states, notably in Africa, at independence inherited a disintegrated transport network with roads and railways pointing like fingers from the harbours to the interior of the country. The network was inadequate for development of an independent economy integrating all regions of the country.

Post-independence

After independence significant investments were made by the respective governments and international aid and financing agencies in rehabilitating existing and constructing new trunk roads. The focus on trunk roads was related to the prevailing development strategy of that period, the single-factor strategy or modernization strategy, which aimed at rapid growth through industrial development.

Trunk roads

By way of example the International Development Association (IDA) during the 1960s spent 30% of its total investments in developing countries on transport infrastructure, notably trunk roads. To cite another example, the length of the networks of paved roads in Kenya was expanded from about 1000 km in 1960 to about 3000 km in 1970.

Rural access roads

During the 1970s the focus of the development policies was gradually turned towards basic human needs and the agricultural sector. As a consequence of the basic human needs and the self-reliance strategies, a larger proportion of the road investments was directed towards rural access roads. The aim was to stimulate agricultural production and improve the conditions of rural life.

During the 1970s and 1980s many African countries initiated large-scale rural access road programmes in order to improve the mobility of the rural population and stimulate the farmers' integration in the market economy. The self-reliance strategy was now supplemented by the economic liberalization strategy.

Maintenance

Aid and financing agencies funding road construction often left it to the recipient countries to maintain the roads. Due to inadequate local institutions and procedures and lack of government funds road maintenance was – and often still is – inadequate.

It has been estimated that the current maintenance budget for trunk roads in Tanzania makes up only 35% of what is required to keep the roads in a satisfactory condition. Secondary and access roads in rural areas are receiving virtually no maintenance funds.

Over the last two decades, however, there has been an increasing awareness of the necessity of adequate road maintenance. Although this awareness has not been fully reflected in actual investment strategies, some improvements have taken place. Road construction and rehabilitation projects now often include a maintenance component. Moreover, purely maintenance-oriented road programmes have been initiated in a number of countries.

Sustainability

Sustainability has become a key word of development organizations during the 1980s and structural adjustment programmes are also influencing the public investment and recurrent budgets. Attention is given to the preservation and continued upkeep of existing facilities rather than the provision of new ones. According to this strategy only roads and other infrastructural facilities that are

deemed politically, economically and institutionally sustainable in a long-term perspective will be taken care of.

Improving road maintenance includes providing sufficient recurrent funds, instituting suitable techniques and establishing sustainable local institutions. This means that the planning and implementation of road projects in the 1990s will require assistance from a wider range of professions than hitherto.

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PART 2

Highway Planning



Motorized camel in Saudi Arabia. (Polfoto)

2 Planning methods

Knud Rask Overgaard, Hoff & Overgaard

2.1 OBJECTIVES, POLICY AND PLANNING

Development objectives

The development of the future highway system should be planned in such a way that it will support the general development objectives of the country, including, for example, economic growth, uniform regional development and full employment. Based on these general objectives more specific objectives for the transport sector may be developed. The means to achieve these objectives comprise transport policy means and indirect means.

Transport objectives

Figure 2.1 is an example of a transport policy model, showing the relationships between general objectives of the society, transport objectives and possible means which may be employed to achieve the objectives.

The broad objectives of a society are normally fairly clear to the government and the politicians and known in general by the population. The objectives of the transport policy, however, are often not very well defined and may have to be deduced from past actions if one wants to obtain a better understanding of the policy being pursued than what may be learned from phrases such as, for example, inexpensive, fast, reliable and safe transport.

Thus transport planning and policy formulation very often interact in a process by which the planners, based on previous experiences with the decision-makers and their preferences, design alternative courses of action and elaborate the likely consequences as a basis on which the politicians then establish objectives and select means that will ensure a desirable influence on the general objectives of the society.

Policy considerations

In developing countries it is of particular importance that the introduction of motorized transportation is based on a sustainable strategy, which will facilitate a gradual improvement of the level of service (ref. 1).

The transport policy must take account of both social and economic considerations. When an area is first supplied with motorized transport, the most relevant question with regard to transport policy is to decide on which mode of transport and which standard should be provided. This will be based primarily on social and financial considerations.

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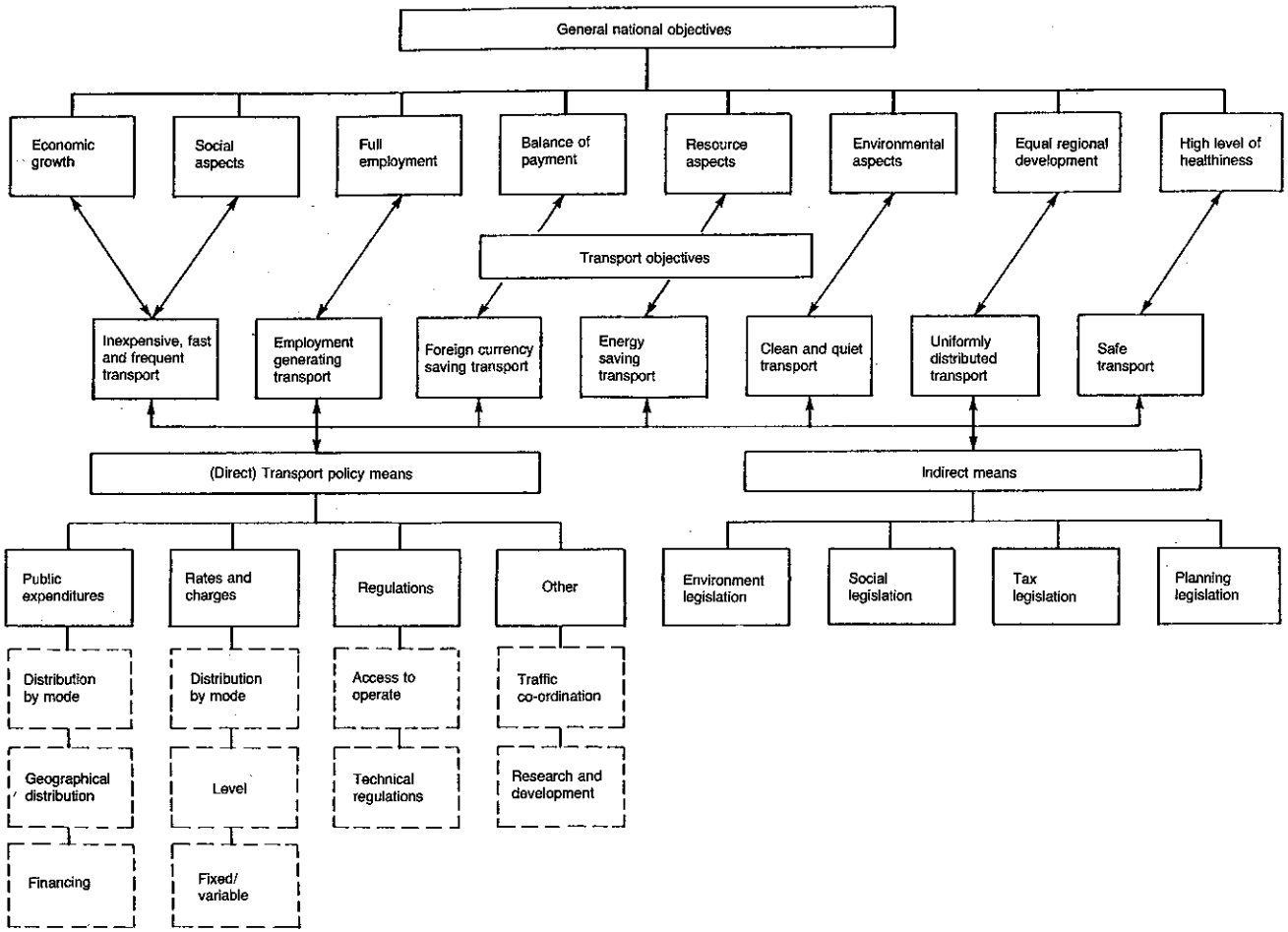


Figure 2.1 Transport policy model.

Later, when the economic development has created a more differentiated demand, and thus a basis for introduction of different modes of transport and services, the socio-economic and the financial feasibility of projects become important for the development of the transport system.

Even later, when population centres develop, the economy improves, traffic congestion emerges, and the awareness of safety and environmental aspects increases, other transport policy aspects have to be given consideration by society in order to decide on the proper balance between the service level for the traffic and the negative consequences associated herewith.

Problems

Some of the most important problems within the field of transport policy and road system development may be summarized as follows:

- How much should be invested in the public transport infrastructure, and to what extent should the level of investment be determined by the economic rate of return and by social/regional considerations?
- How should public transport investments be divided between roads, railways, ports, airports, etc.?
- Should emphasis be on primary, secondary or tertiary roads?
- What kind of road projects should be given priority, e.g. those increasing capacity, those reducing costs, those increasing the level of service, those improving safety, or those improving the environment?
- How should financial allocations be divided between new construction, rehabilitation and maintenance of roads?

Planning

The answers to these problems are often elaborated in the form of a road system development plan, comprising a long-term strategic plan, a medium-term development programme and an annual works programme, which is updated every year. The following sections present a methodology for elaboration of a highway network development plan and programme.

2.2 ELABORATION AND EVALUATION OF ALTERNATIVE NETWORKS

Network elaboration

As the highway plan for the future originates in the existing road network, this should be inventoried with regard to extent, standard, condition, utilization, etc., as a starting point for the planning of the future system. Other important inputs for the elaboration of an appropriate long-term highway development plan are the relevant objectives and policies established by the society, the expected future transport demands, and the available resources (primarily finances).

Gradual improvements

However, even when all these inputs are known there is no definite method available for elaborating the best highway system in the long term. The procedure usually adopted consists of a more or less systematized learning process, by which an initial network proposal is gradually improved on the basis of comparisons of the consequences of marginal network changes.

Intermodal competition

This 'fine-tuning' may be preceded by a comparison of a small number of widely different system proposals if considered expedient. Especially when strong intermodal competition has to be considered this approach may be used to establish the approximate demand for road transport as a basis for 'fine-tuning' the highway development plan.

Maximum network

Due to the uncertainties associated with traffic forecasting for developing countries, and to the long-term nature of road investments, the highway development plan probably should not cover more than a 10–15 year period. The maximum conceivable network at the end of this period may be established by adding the following four types of projects to the existing network:

- current and committed projects including those for which a construction contract has been signed;
- projects that are natural extensions of existing and committed projects to complete a road route or complete the basic structure of the road network;
- projects that are considered important from the development or strategic point of view;
- other technically feasible projects that have potential economic value but are not essential to the structure of the road network.

Candidate roads

In order to identify candidate roads for the plan it may be advisable to establish a map indicating major transport desire lines between the future main activity areas (Figure 2.2). By comparing these corridors of potential high transport demands with the existing network configuration, proposals for new highway links may emerge (Figure 2.3).

By assigning the estimated future transport demand (Section 2.4) to the existing highway network it may be possible to identify *inter alia* corridors which will require capacity increases and links which will require upgrading in the future.

Highway standards

The highway standards to be applied to the potential new networks should take account of the expected future traffic volumes as minimization of total transport costs is likely to be an important objective of the highway development plan. Special consideration should be given to the fixing of the legal axle load as this will influence initial pavement costs as well as future maintenance costs, which will become more important as time passes.

Incremental analysis

The network consisting of the existing roads and the current and committed projects serves as the basis for the incremental analyses by which candidate projects from the maximum network are gradually added to the basic network.

Economic feasibility

The feasibility of incremental expansions of the network are tested by comparing road user savings to the total construction and maintenance costs pertaining to the same expansions. In this way the optimal plan, as well as important indications on how the network should be developed in terms of priorities, may be determined.

2.3 HIGHWAY DEVELOPMENT PLAN AND PROGRAMME

Plan and programme

After the optimal long-term plan for the highway system has been established on the basis of general development objectives, expected future transport demands and economic evaluation of alternative highway networks, a programme for the implementation of the plan has to be elaborated, as the resources, and in particular the finances, available to the highway sector in developing countries are always insufficient to cope with the needs for road development and maintenance. Therefore it is necessary to select some of the projects needed to complete the long-term plan for inclusion in a five-year development programme and annual work programmes to be elaborated subsequently.

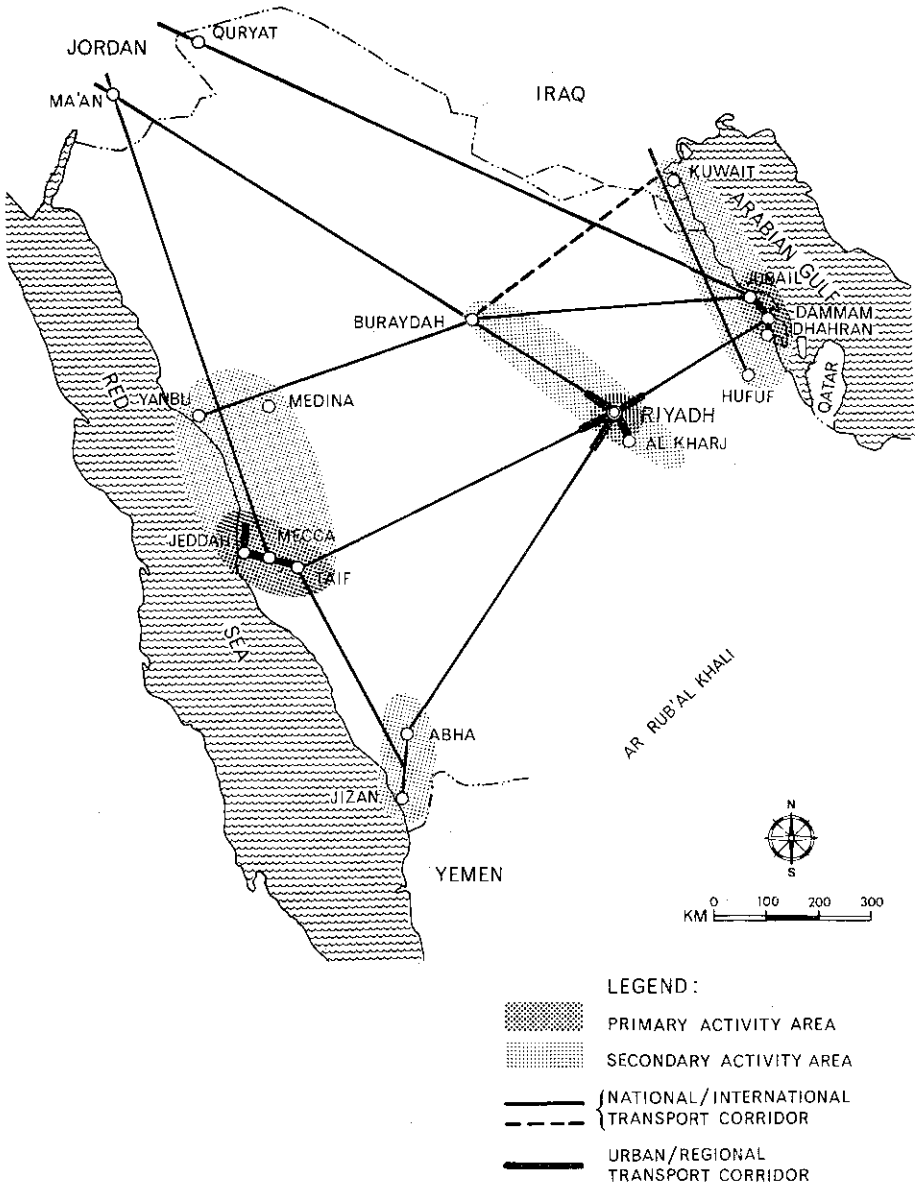


Figure 2.2 Main activity areas and major transport desire lines.

Analysis incomplete

Unfortunately the network analysis described in Section 2.2 is only able to provide some of the answers required for establishing highway development programmes which will reflect a proper balance between the various general objectives of the society and consequently be acceptable to the decision-makers.

The network analysis focuses on the economic aspects of primary and secondary roads, but as suggested above the road classes perform different functions, some of which are directly related to transport costs whereas, for example, the main

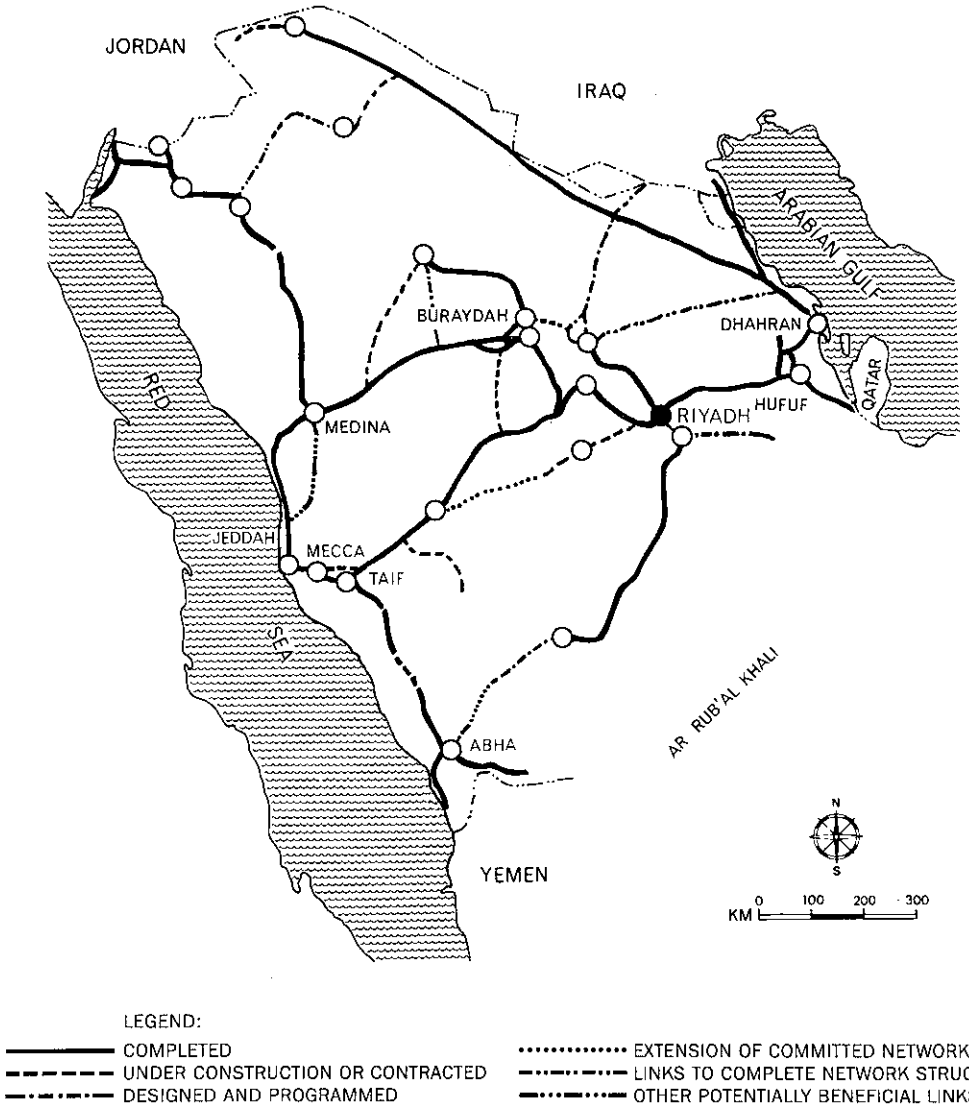


Figure 2.3 Existing and potential new highway links.

purpose of tertiary roads is to provide access to adjacent land for motorized vehicles and thus to a great extent to serve social purposes. The planning of tertiary roads is discussed in Chapter 3.

Additional consequences

Many other consequences of road construction, improvement and maintenance are also difficult to quantify fully in economic terms due to lack of proper definitions, problems of measurement or absence of market prices, generally accepted shadow prices (see the definition in Chapter 3) or politically determined trade-offs between different consequences such as, for example, transport costs, travel speeds, accidents, pollution, etc.

The fact that the consequences of improvements to the various highway classes do not affect different population groups in the same way adds further complications to the problem of defining the best possible road development programme within given budget constraints.

Adjustment of standards

Thus it may, for example, be necessary to strike a balance between overall objectives, such as economic growth and efficiency, and regional and individual equity. In Figure 2.4 curve 1 shows for different traffic volumes (or road classes) the level of service (or the road standard) which would be the most economic one for society if no budget constraints existed. Curve 2 represents a situation where a minimum level of service has been fixed for roads with low traffic volumes and the available budget is insufficient to also provide the high level of service which would be economic for roads with high traffic volumes.

Over-investment

It is important to avoid over-investing in particular road classes or in specific geographical regions by introducing specific road standards or road network densities at times when they are not yet justified by traffic levels as this may deprive other road links or areas of needed improvements.

Stage construction

Therefore it is important to select from the long-term highway plan for early implementation those projects within each road class which provide the highest returns, and to consider the possibilities of improving the project returns by construction in stages. In this respect the proper timing of paving the roads is of particular importance.

HDM-III

This question may be studied by means of the Highway Design and Maintenance Standards Model, version 3 (HDM-III) described in Chapter 4. HDM-III will provide a complete programme for optimal treatment of a road pavement throughout its lifetime given appropriate inputs on initial road condition, maintenance and

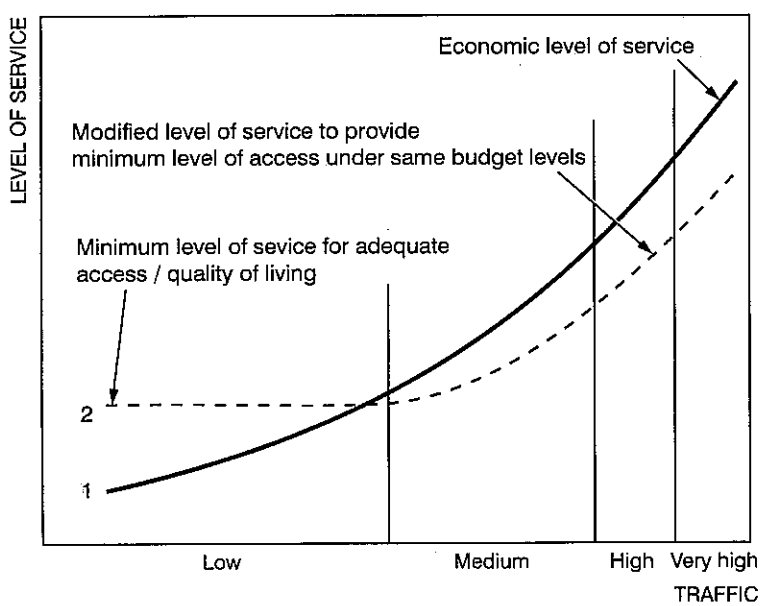


Figure 2.4 Effect of equity objective on economic efficiency standard.

improvement alternatives and costs, traffic volumes and vehicle operating costs, etc.

The HDM-III model requires as an input *inter alia* traffic volumes on a link basis and therefore does not take account of traffic diversions unless manifest in the input to the model.

Application of HDM-III

The calculations may, of course, be carried out for all links of a network and aggregated to provide information on total maintenance and improvement works to be carried out on an annual basis and their associated costs. However, at the level of highway system planning, where detailed and precise data usually are not available, the HDM-III model may more appropriately be used to analyse the performance of representative links during the chosen study period in order to establish proper investment and maintenance cost profiles for system evaluation and budgeting purposes.

Budget allocation

As the consequences of different highway investments and maintenance activities cannot all be compared on the basis of a common denominator the total budget available for maintaining and improving the highway system in a certain period of time will have to be allocated to different types of activities, such as, for example, new construction, upgrading, rehabilitation and maintenance of roads by class, by political decision-making.

However, this process should take into account the overall national development objectives, the analyses carried out and the requirements and justifications consequently established for each type of work as well as practical considerations concerning, for example, implementation capacity, so that the programme will have a realistic basis and can serve as a proper guideline for future implementation and not only as a wishful target.

Budget requirements

Although there will always be a limit to the budget that can be made available to the highway sector it is important also to establish the funding needed over a period of, say, five years to implement all economically viable projects, as such a budget would demonstrate the real needs of the highway sector and the returns which the society could expect from investments in this sector as compared to other sectors.

Investment programme

Figure 2.5 shows an example of the investment programme required to implement a 15-year roads development plan. (Maintenance and rehabilitation costs are not shown.) The programme is divided into three five-year periods with the first one being further subdivided into one-year periods. The current year is included for comparison.

The budget is divided into separate budget items for primary/secondary and tertiary roads respectively, the budget for each of these two road classes being composed of one part to cover investments in specific projects identified by a planning study and another part to cater for additional projects, which may be identified later.

It should be noted that only a small number of primary and secondary roads is expected to be added to the plan later, whereas a larger share of the tertiary roads are still unidentified. Furthermore, this share shows an increasing tendency towards the end of the planning period.

Road management system

In order to be able to improve the planning, programming and budgeting procedures and to currently update five-year programmes and annual works plans and budgets as required, for example, due to additional aid funds being made

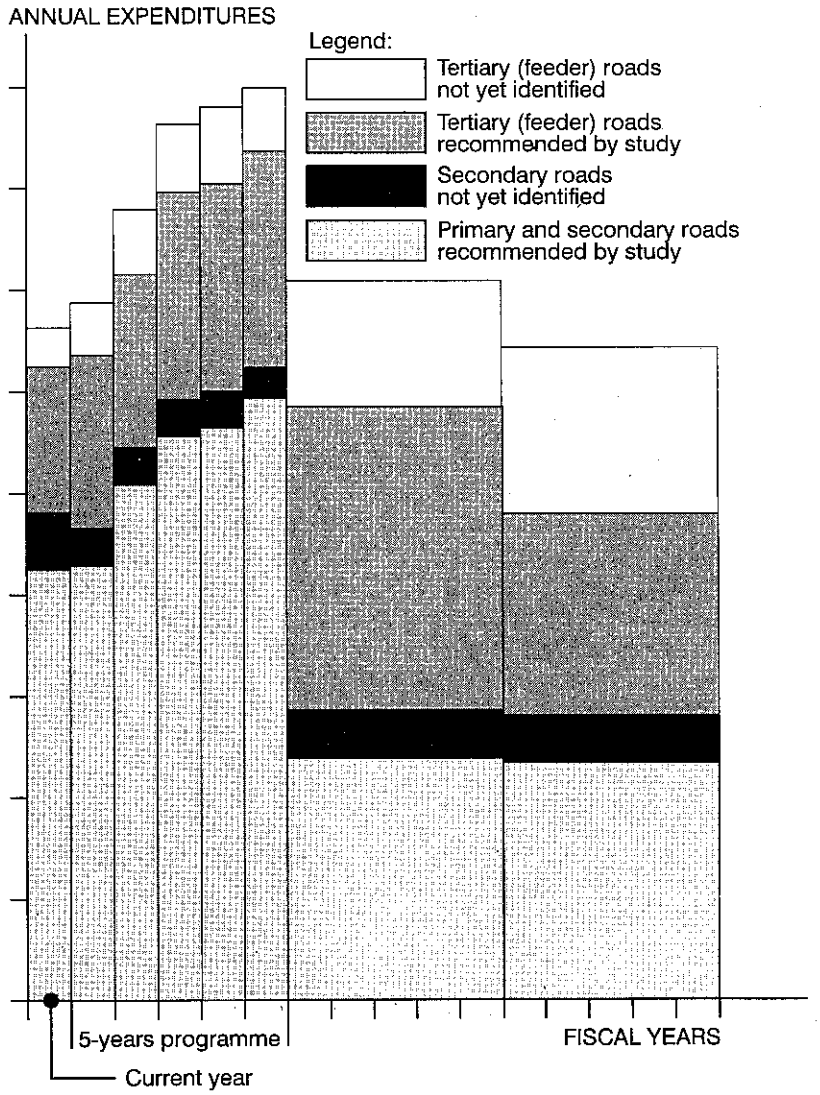


Figure 2.5 Roads investment programme.

available, some developing countries are now attempting to develop computerized integrated road management systems. Some general requirements established for one such system are the following:

- The major system requirement is to ensure that investments in the interurban road system are allocated to programmes, regions and links to produce the maximum economic growth throughout the nation. This can be achieved through the application of sound economic principles to the investment process.
- Another system requirement is related to the national objective of regional and individual equity. There is a tension here with the requirement of maximum

economic growth but this can be satisfied by the establishment of minimum serviceability levels.

Revisions

- A major problem which has to be addressed in this system is frequent major fluctuations in budgets and policies which necessitate major programme changes to be carried out in short periods. The integrated system must be capable of producing revised plans, programmes and contract packages at very short notice. This requirement will necessitate that all systems are both integrated and computerized.
- Another general requirement is that the plans and programmes must be economically justifiable so that financing from international lending agencies for each programme can be obtained. It is essential that the economics of analysing the various investment programmes by the system be based on a model such as the HDM-III model. However, the model must be suitably adapted and calibrated for specific conditions.
- Given that the integrated system will be used by all provincial offices it is essential that it be easy to operate. To meet this requirement the system should be menu driven and based on microcomputers rather than a mainframe computer.
- The integrated system must only be used as a guide and tool in setting programmes as there are often national objectives or provincial priorities which will take precedence over the programme determined by this system. Flexibility to accommodate these changes in the process is necessary.

2.4 TRANSPORT DEMAND FORECASTING

Corridors and networks

In order to prepare optimal designs for improved or new highways or to make efficient plans for development of highway networks it is necessary to have information on current traffic and estimates of future traffic.

In the following, forecasting of future traffic on individual highways is discussed first, and the more comprehensive estimation of future traffic volumes on the interdependent links of a highway network is presented next.

2.4.1 Traffic forecasting for individual highways

Corridor forecasts

When the influence of a highway improvement or a new highway is confined to a narrow highway corridor it is sufficient to base traffic forecasts on the existing highway traffic (in the case of an improvement), and the traffic which may be diverted from the immediate parallel highways upon opening of the improved or new highway.

Traffic components

The existing and the diverted traffic together are considered to be the current traffic. The future traffic further comprises normal traffic growth, generated traffic and development traffic (Figure 2.6).

Growth curves

If the future economic development is expected to continue as in the past then the normal traffic growth may be estimated by extrapolation of time series data on traffic volumes on the existing and/or the parallel highways. The following growth curves are commonly used:

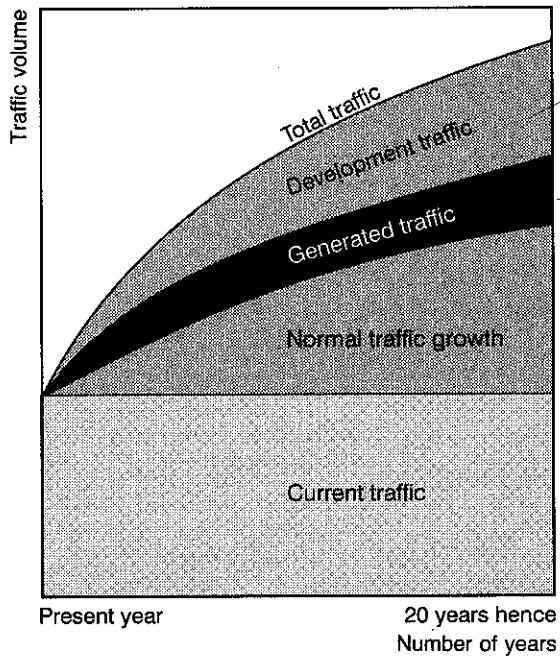


Figure 2.6 Traffic development on an improved road.

$$\text{Linear growth:} \quad y = a + b \cdot t;$$

$$\text{Exponential growth:} \quad y = c \cdot e^{dt};$$

$$\text{Logistic growth:} \quad y = \frac{Y_s}{1 + f \cdot e^{-gt}};$$

where:

y = traffic volume;
t = time variable (e.g. number of years after base year);
e = base of natural logarithms (i.e. 2.7183);
Y_s = saturation level (asymptote);
a, b, c, d, f, g = parameters, which may be developed from time series data (or otherwise established).

Traffic and GDP

As an alternative to extrapolating historical traffic data, traffic growth can be related linearly to Gross Domestic Product (GDP). This is preferable since it explicitly takes into account changes in overall economic activity. However, using this relationship for forecasting requires a realistic forecast of GDP, which is not always available.

If it is thought that a particular component of the traffic will grow at a different rate from the rest, then it should be specifically identified and dealt with separately.

Generated traffic

Generated traffic arises because a journey becomes more attractive due to the new or improved road. It is only likely to be significant in those cases where the road investment brings about large reductions in transport costs.

In the case of a small improvement within an already developed highway system, generated traffic will be small and can normally be ignored. Similarly, for projects involving the improvement of short lengths of rural roads and tracks, there will usually be little generated traffic. However, in the case of a new road allowing access to a hitherto undeveloped area, there could be large reductions in transport costs as a result of changing mode from head-loading to motor vehicle transport and, in this case, generated traffic could be the main component of future traffic flow.

Development traffic

Development traffic consists of the extra trips to areas where the development has been speeded up due to the new or improved highway. This traffic, which usually develops over a long period of time, depends upon the extent to which authorities allow development to take place, and the actual land usage alongside the highway, as the various activities give rise to different traffic flows.

In practice it is difficult to distinguish between generated traffic and development traffic, and the two are usually taken together and termed generated traffic.

Price elasticity

Demand relationships can be used for forecasting generated traffic. The price elasticity of demand for transport measures the responsiveness of traffic to a change in transport costs, for instance following road investment. The price elasticity is defined as the ratio of the percentage change in travel demand to the percentage change in travel cost. Evidence from studies in developing countries give a range of between -0.6 to -2.0 for the price elasticity of demand for transport, with an average of about -1.0. This means that a 1% decrease in transport costs leads to a 1% increase in traffic (ref. 2).

The available evidence suggests that the elasticity of demand for passenger travel is usually slightly greater than the average of about -1. In general, the elasticity of demand for goods transport is much lower (-0.1 to -0.5) and depends on the proportion of transport costs in the commodity price.

Door-to-door travel costs

Calculations should be based on door-to-door travel costs and not just on that part of the costs incurred on the road under study. Generally, this implies that the reduction in travel costs and increase in traffic will be smaller than measurements on the road alone suggest.

In some cases it will be more appropriate to forecast generated traffic on rural access roads on the basis of the anticipated response of farmers to road improvements. This so-called 'producer surplus' approach is described in Chapter 3.

2.4.2 Traffic forecasting for highway networks

In order to plan the future development of the highway network in an optimal way it is necessary to assess the future transport demands between the various parts of the area under study and to estimate the possible future traffic volumes likely to use alternative preconceived highway networks.

Traffic matrix

For this purpose the area is divided into zones, which generate/attract the traffic. Table 2.1 shows how the transport demands of an area divided into n zones are expressed in the form of a traffic distribution matrix, where:

T_{i-j} = the number of trips generated by zone i and attracted by zone j;
G_i = the total number of trips generated by zone i;
A_j = the total number of trips attracted by zone j.

Table 2.1 Matrix for trips between n zones.

From zone	*To zone* 1	·	j	·	n	$\sum_j T_{i-j}$
1	$T_{1\text{-}1}$	·	$T_{1\text{-}j}$	·	$T_{1\text{-}n}$	G_1
·	·	·	·	·	·	·
i	$T_{i\text{-}1}$	·	$T_{i\text{-}j}$	·	$T_{i\text{-}n}$	G_i
·	·	·	·	·	·	·
n	$T_{n\text{-}1}$	·	$T_{n\text{-}j}$	·	$T_{n\text{-}n}$	G_n
$\sum_i T_{i-j}$	A_1	·	A_j	·	A_n	$\sum_{i,j} T_{i-j}$

Different matrices may be used to express the future transport demand or part of it such as, for example, total passenger trips by all modes, car trips for all or specific travel purposes, total goods volumes by all modes, and truck trips with all or selected commodities.

Growth factor model

If the existing trip distribution in terms of an origin–destination (O-D) matrix has been established, e.g. by means of traffic surveys or application of a 'synthetic' model with subsequent calibration against traffic counts (ref. 3), future trip matrices may be developed by using a growth factor model.

The growth in traffic generated/attracted by each zone is estimated on the basis of an analysis of the expected development of traffic generating sources. For each zone separate growth factors may be elaborated for generated and attracted traffic. However, if the O-D matrix for existing traffic does not distinguish between traffic generated and traffic attracted by a zone, a composite growth factor (F) will have to be established for each zone. In this case the average growth factor model estimates the future traffic (T_{ij}) between two zones i and j starting from the formula:

$$T_{ij} = t_{ij}\frac{F_i + F_j}{2}$$

where t_{ij} is the observed traffic and F_i and F_j the growth factors for total traffic from/to the zones i and j respectively. The calculations usually require an iterative procedure to balance the system (ref. 4).

The average growth factor model does not explicitly take into account future changes in the access between the various zones. Therefore it is more applicable to situations with limited or uniform transport improvements than to major 'spot' improvements of the highway network.

Growth factor

The passenger traffic growth factor of a zone may, for example, be calculated as follows:

$$F = \left(\left(\frac{I \cdot E}{100} + 1\right)\mathrm{CP}\right)^n$$

where:

F = growth factor for total passenger traffic to/from the zone;
I = annual growth rate (%) for per capita income in constant prices;
E = passenger transport demand–income elasticity;

CP = compound population growth rate per year (e.g. 1.03 if population grows by 3% per year);
n = number of years between survey year and forecast year.

Applicability

As indicated above, the growth factor model is mainly applicable to short- and medium-term forecasts. Therefore, variations in I, E and CP over time should be taken into account for forecast periods of say more than 10 years (i.e. $n > 10$) by calculating F in two or more steps as relevant, if a growth factor model is to be employed.

For medium- and long-term forecasting the gravity model presented in the following may be a more relevant tool due to its ability to better reflect changes in geographic development patterns and transport accessibility.

If modal competition is significant in the study area the modal split may have to be taken into direct consideration when estimating future road traffic volumes between zones. Finally, these volumes should be allocated (i.e. traffic assignment) to road sections in the network under study in order to estimate future traffic flows.

Four-steps model

The above description indicates a sequential traffic forecasting model consisting of the following four steps or submodels:

- traffic generation and attraction by zone;
- traffic distribution (matrix estimation);
- modal split (when required);
- traffic assignment.

Due to its relative simplicity, widespread application and educational advantages, this model, which is actually two models, i.e. one for passenger transport and one for goods transport, will be used as a reference in the following. Other models combine some of the four steps (ref. 4).

2.4.3 Passenger transport forecasting

Model development

The estimation of the future demand for passenger transport is usually based on an analysis of the existing travel behaviour, which results in models that explain passenger trip-making in relation to socio-economic variables such as, for example, demographic factors, personal incomes, transport demand–income elasticities, etc. (The elasticity is 1 if a 1% increase in income results in a 1% growth in traffic.) If it is not possible to collect local data on trip propensities it may be necessary to use external trip rates applicable to the situation in question.

Initially it is necessary to decide whether road traffic may be estimated independently or competition from other modes will have to be taken explicitly into account. When road traffic can be forecast directly, the estimate is often worked out for cars or vehicles, i.e. including buses and para-transit types of vehicles, instead of passengers who subsequently would be converted to vehicles by means of average occupancy rates.

As trips for different purposes exhibit different distribution patterns and growth rates it is also necessary to decide if all trips may be handled together or a division by purpose, such as, for example, business trips and private trips, will have to be applied.

Trip generation

The number of passenger trips generated/attracted by a zone may be estimated as the population of the zone times a trip rate. A more advanced method distinguishes between urban and rural populations and applies different trip rates to the two groups. The trip rates usually also vary with real income per capita. This must be taken into account when the future traffic generation/attraction by zone is calculated, e.g. by applying appropriate transport demand–income elasticities and economic growth assumptions to adjust the observed trip rates. Elasticities between 1 and 2 depending on *inter alia* the degree of development are common.

Traffic distribution

The number of passenger trips between the zones may be calculated by means of a restrained gravity model:

$$T_{ij} = a_i \cdot G_i \cdot b_j \cdot A_j \cdot f(d_{ij})$$

where:

T_{ij} = number of trips generated by zone i and attracted by zone j;
G_i/A_j = total trip generation/attraction by zone i/j;
$f(d_{ij})$ = a function which expresses the resistance that has to be overcome to travel between the zones i and j, e.g. $f(d_{ij}) = d_{ij}^{\alpha}$ where d_{ij} is the travel time between the zones i and j and α is a parameter estimated on the basis of an analysis of the present transport supply and the observed trip distribution;
a_i, b_j = calibration factors which will ensure a trip distribution matrix where the row and the column sums correspond to zonal trip generations and trip attractions respectively.

Thus the future trip distribution has to be calculated by applying an iterative procedure (ref. 4).

Modal split

If the trip matrix includes trips by more than one mode a modal split calculation will have to be performed for zone pairs being served by more than one relevant mode. When two modes only are competing a modal split model relating modal split percentages to transport cost or time ratios as shown in Figure 2.7 may be sufficient.

Public transport

If more public transport modes are available between two zones, traffic may be divided between these modes on the basis of the following expression:

$$T_m = k\frac{f_m}{c_m \cdot t_m}$$

where:

T_m = traffic by mode m;
f_m = frequency of mode m;
c_m = transport fare for mode m;
t_m = travel time for mode m;
k = parameter which ensures that traffic by all modes corresponds to total traffic between the two zones.

Logit model

The logit model may determine mode and route distribution simultaneously. Furthermore, this model is usually based on individual travel behaviour and not on average figures for zones. However, the development of the model is somewhat

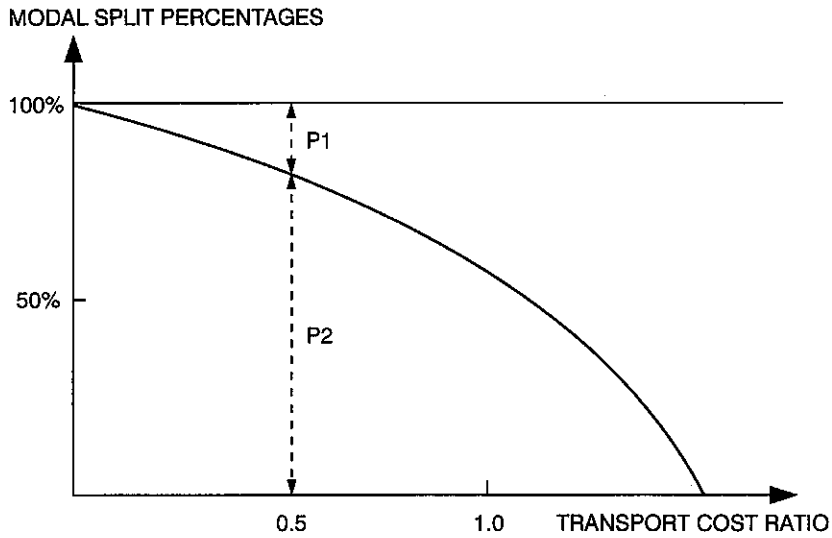


Figure 2.7 Modal split model.

complicated and requires data from passenger interviews and detailed data on relevant transport alternatives (ref. 5).

Traffic assignment

The future traffic matrix may be assigned to the highway network alternatives to be evaluated using, for example, the all-or-nothing principle, which allocates all traffic between two zones to the optimum route, or multiple route assignment (ref. 4).

The optimum route may be, for example, the fastest or the cheapest route or be based on a combination of travel time t and distance d, i.e.

$$a \cdot t + b \cdot d$$

where a and b are parameters estimated from observed route choices.

Travel time components

The travel time should include all components of the door-to-door trip. Thus for a journey by public transport the travel time may, for example, be calculated as follows:

$$t = a \cdot t_1 + b \cdot t_2 + t_3 + c \cdot t_4 + t_5 + d \cdot t_6$$

where:

t = door-to-door weighted travel time;
t_1, t_6 = time walked to/from public transport;
t_2 = time spent waiting;
t_3, t_5 = travel time in public transport vehicles;
t_4 = transfer time between routes;
a, b, c, d = parameters which express the inconvenience travellers experience in connection with the various time components. Normally the parameters are higher than 1. They may be estimated from observed route (or mode) choices.

Example

Figure 2.8 presents a possible methodology for passenger traffic forecasting

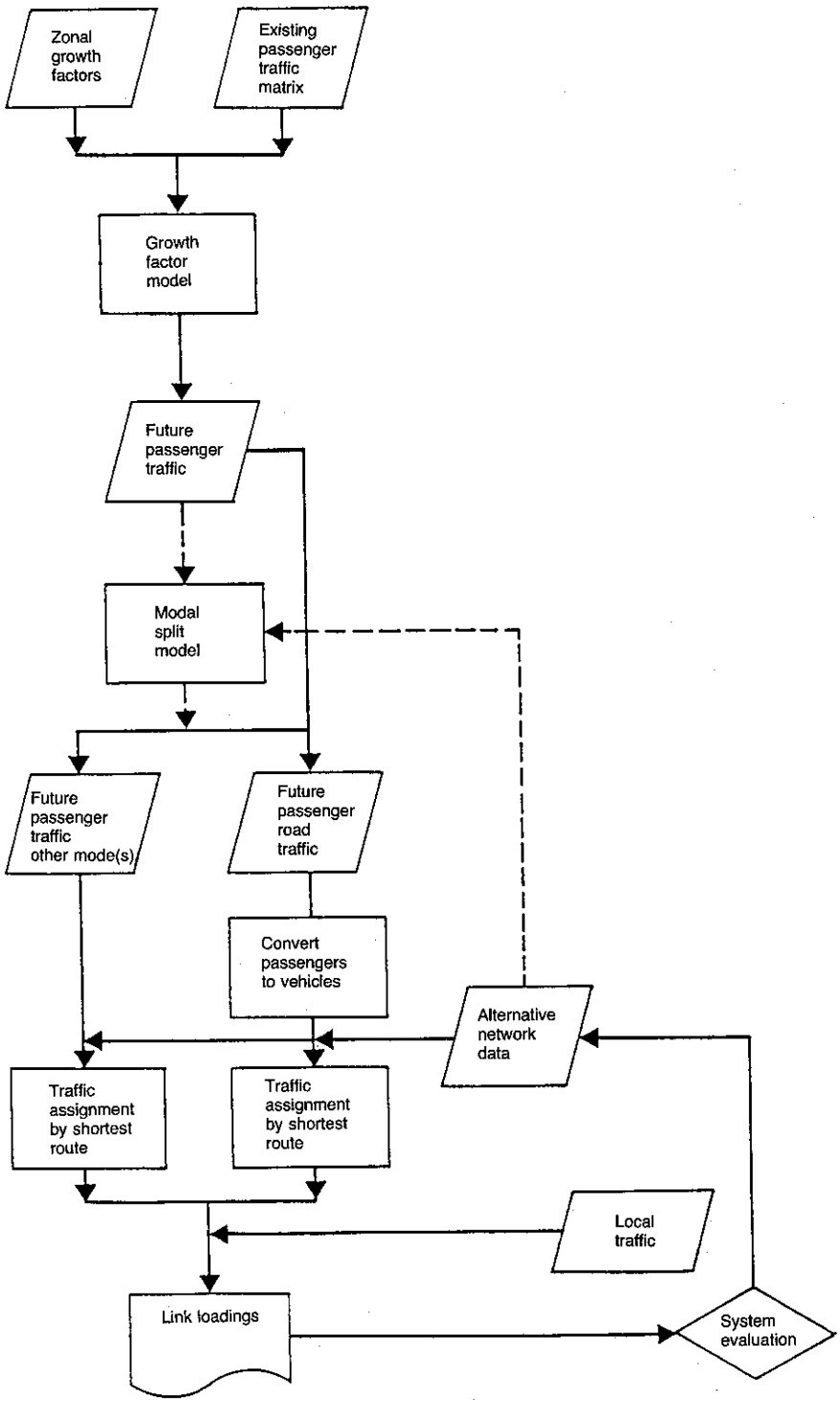


Figure 2.8 Methodology for passenger traffic forecasting and system evaluation.

which applies the growth factor model. The methodology may include or exclude a modal split calculation as relevant. It may also be applied directly to car or passenger vehicle traffic. Traffic assignment is accomplished by means of the all-or-nothing principle using fastest (shortest) routes. Local road traffic must be added to the interzonal flows assigned to each link and taken into account when alternative road networks are evaluated.

2.4.4 Goods transport forecasting

Commodity classes

Due to different transport characteristics and growth and distribution patterns, commodities should be classified into homogeneous groups such as, for example, agricultural products, petroleum products, building and construction materials, industrial products, consumer products, etc. Usually less than 10 groups will suffice, and sometimes only total truck traffic is considered.

Traffic generation

The traffic generation/attraction of each zone is estimated on the basis of the expected future surplus/deficit of each commodity by zone. The surplus equals production (plus import) less local consumption within the zone, and the deficit equals consumption (plus export) less the amount supplied locally. Thus the surplus/deficit figures indicate the tonnages to be transported between the zones (intrazonal traffic has to be accounted for separately).

The estimation is accomplished by analysis of the regional economy, including historical growth rates and future growth potential taking account of land usage, plans for agricultural, industrial and construction projects, employment, population and personal income development, propensities to consume, etc.

Traffic distribution

Models corresponding to those described for passenger traffic may also be used to estimate future goods traffic distribution, modal split and route assignment. However, if the various transport modes are combined into one network with interchange possibilities the goods traffic distribution and modal split may be achieved simultaneously by means of the transportation model, which is a special type of linear programming model (ref. 6).

Transportation model

The model, which reflects intermodal competition, matches for each commodity the surplus/deficit of the zones in such a way that total transport costs are minimized, considering the minimum cost mode and route between each pair of zones. The least cost mode/routes are found by applying a 'shortest' route algorithm to the multi-modal transport system. Assignment may be completed as before by means of the all-or-nothing principle or using multiple routes.

Rural access

In connection with assessments of alternative transport modes for provision of rural access, combinations of transport infrastructure improvements, transport aids and storage facilities should be taken into consideration. Local storage facilities dampen transport demand peaks and consolidate local transport demand (ref. 7).

2.5 PRACTICAL CONSIDERATIONS

The planning methodology to be applied in a particular country should be adapted to the actual conditions and problems at the time of planning (ref. 8).

Data needs

The data needed for quantitative planning of a road system is usually not readily

available in developing countries. Thus the first approach to national road planning on an analytical basis may have to be based on rough and partly estimated data.

'Estimating future traffic is still an imprecise – but essential – art. Since transport investments commonly have long lives, decisions to make such investments inherently involve long-term forecasts. It is clearly preferable to be explicit about the underlying assumptions than to leave them unstated' (ref. 9).

The analytical approach often highlights problems which are otherwise neglected, and it gives rise to a usually much needed systematic collection of data concerning roads and traffic. The advance of Geographical Information Systems (GIS) is expected to facilitate this process in the future (ref. 10).

Framework

The national road plan represents a framework for more detailed plans and studies, including feasibility studies, design and implementation of specific road projects, and for the co-ordinated implementation of other activities contained in the plan.

Updating

It is emphasized that national road planning must be a continuous process with current recording of developments and updating of plans as the basic conditions change.

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3 Economic evaluation

Ole Møller, The Danish Transport Council

3.1 INTRODUCTION

Purpose

This chapter deals with methods for ex-ante analysis of the economic effects of road investments. Economic evaluation is part of the project cycle that most donor funded investment projects undergo, cf. Chapter 23. The purpose of the evaluation is to produce information on the basis of which investment decisions can be made. Economic evaluation is normally used in feasibility studies as well as in appraisals (the first two phases of the project preparation stage).

Often the economic evaluation includes an analysis of the net contribution that the investment will make to the society as a whole. Thus, the economic analysis differs from that which would be undertaken by private companies appraising commercial investments.

Evaluation method

Selecting the appropriate evaluation method and level of detail should be determined by:

- the type of decision problem;
- the type of project;
- the objectives of the project; and
- the time, economic resources, skills and data available.

Decision problems

Typical decision problems regarding road investments are:

- deciding the design standard or alignment of a road;
- determining the optimum timing of an investment;
- priority ranking a group of mutually independent road projects.

Type of project

The type of project should be considered before an economic analysis is carried out. The type of project will have an important bearing on the engineering solutions and the benefits to be included. A distinction can be made between:

- new construction and upgrading (improvement to higher standard);
- reconstruction and rehabilitation (to original standard);
- stage construction (planned improvement at fixed stages);
- maintenance projects.

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The objectives of a project must be reflected by the benefits to be included in the analysis. While the objective of interurban trunk road projects normally is to reduce vehicle operating costs and road maintenance costs, the objectives of rural access road projects may include improving access to social services, stimulating agricultural production, etc. The benefits of urban roads may include significant time and accident savings and savings in fuel consumption from alleviating congestion. The special aspects of evaluating urban road investments will not be addressed in this chapter.

Road type

Most highway investments will require a cost–benefit analysis as outlined in Sections 3.2–3.4 below. A detailed cost–benefit analysis, however, is normally neither justified nor suitable for rural access road investments. Rural access roads have relatively low construction costs per km and they are often expected to affect the local economy and social life. The special aspects of evaluating rural access roads are addressed in Section 3.5.

Analysis period

When conducting an economic analysis of a highway investment, the length of the analysis period must be determined at an early stage.

Long analysis periods are useful when comparing mutually exclusive projects. Short analysis periods may be appropriate for small projects where the life of the investment is expected to be limited to a few years.

The difficulties in making traffic forecasts, etc., for long periods into the future argue against very long analysis periods. Whatever time period is chosen, the project will usually have some residual value at the end of this period. The residual value can be approximated as the difference in cost between rebuilding the road at the end of the period using the structure remaining and the construction cost if the initial project was not implemented.

For most road projects an analysis period of 15 years from the date of opening is appropriate but this should be tested by the evaluation. Choosing the same analysis period as the design life of the pavement of a paved road simplifies the calculation of residual value.

Investment models

Economic analysis of highway investments entails complicated calculation of vehicle operating costs under a range of conditions and discounting and comparing of all costs and benefits over the analysis period.

Computer models are available for assisting in these calculations. The most well known of these are the Road Transport Investment Model, version 3 (RTIM3) and the Highway Design and Maintenance Standards Model, version 3 (HDM-III). The RTIM3 was developed by the Transport Research Laboratory (TRL) in Great Britain and designed to be simple to use. The HDM-III has been developed by the World Bank and this model is more comprehensive than RTIM3. The HDM-III can be used for evaluating a whole network of roads while the RTIM3 is only able to analyse one road link at a time.

Both models simulate the performance of a road over time and under traffic. Costs and benefits are determined by applying unit rates to quantities that are calculated. Since these unit rates are provided by the user, the models are applicable to a wide range of economic and financial environments. Costs and benefits are finally discounted and compared. The HDM-III is described in Chapter 4.

3.2 COST ESTIMATION

Construction/ maintenance

The costs of a road project consist of the total cash expenditure that will be necessary to complete and operate the project. The two components are referred to as:

- construction/rehabilitation costs;
- maintenance costs.

This section focuses on construction costs, while maintenance costs are discussed in Section 3.3.3 below. Often road rehabilitation projects result in net maintenance cost savings, i.e. a benefit, since the improved road is less costly to maintain than the existing one.

Project stages

The required accuracy of the cost estimate depends on the stage of the project. At the identification stage a very crude estimate will be sufficient, while a refined and accurate estimate will be needed at the preparation stage.

The cost estimate should be sufficiently accurate to:

- give the decision-makers a realistic impression of the resource requirements;
- allow for comparing alternative project solutions.

Estimating techniques range from the broad-brush category of 'global' estimation to the more detailed unit rate technique.

Global estimation

Global estimation describes a simple technique which relies on historical data on costs of similar projects related to the overall size or capacity of the asset provided. Examples of ratios used are:

- cost per km of road;
- cost per square metre of bridge deck.

Using this type of historical data entails dangers. It is not always clear what costs are included. For example, is the cost of design and supervision included? Are tax and duties included?

There is also the risk of not comparing equal projects. Are the levels of quality, pavement thickness, etc., equal? Are terrain and soil conditions comparable?

Unit rate

The unit rate estimation technique is based on the traditional bill of quantity approach to pricing construction work. The bill of quantities contains the quantities of work to be carried out, measured in accordance with an appropriate measurement. Historical data on unit rates or prices is selected for each item in the bill. The data is taken from recent similar contracts or published information, e.g. price books for civil engineering.

When a detailed bill of quantities is not available, quantities will be required for the main items of work and these will be priced using 'rolled up' rates which take account of the associated minor items.

Again, using historical data entails some risks. It is likely that the previous projects were not carried out in identical conditions, using identical construction methods and with the same duration.

It should also be noted that unit rates quoted by contractors in their tenders are not necessarily related directly to the items of work they are pricing. It is common practice for a tenderer to distribute the total funds required across his items in

order to meet objectives such as cash flow and anticipated changes in volume of work.

However, reliable cost estimates can be prepared by experienced estimators with good intuitive judgement and the ability to assess the realistic conditions of the work.

Sensitivity

The cost estimate can be improved by analysing the sensitivity of the total costs to alternative assumptions on key factors affecting the main cost components. In this connection the cost implication of major risks should also be analysed.

3.3 BENEFIT ESTIMATION

Categories

Economic analysis of a road project requires estimates of not only the costs associated with the project but also of the benefits that are expected to occur. Normally the following benefits are considered (ref. 1):

- vehicle operating cost savings;
- road maintenance cost savings;
- time savings by travellers;
- reduction in road accidents;
- economic development benefits.

Different benefit categories will predominate for different types of road projects. Vehicle operating cost savings will normally be the most significant benefit of interurban road projects (rural trunk roads) in situations where the cost of time is low.

Time and accident savings will normally play a significant role in urban road projects in situations with high traffic volumes and high costs of time. For low trafficked rural access roads the economic justification often rests on the expected development benefits in terms of induced agricultural surplus.

The economic development benefit will manifest itself as generated traffic. If development benefits and generated traffic are both being evaluated it is important to avoid double counting.

3.3.1 Traffic demand

The expected volume and composition of the future traffic on the road are key factors influencing the level of all the above-mentioned benefits. When estimating benefits it is necessary to separate traffic into three categories:

- Normal traffic. Traffic which would occur if no investment took place, including normal growth.
- Diverted traffic. Traffic which is attracted from an alternative route (or mode) to the project road.
- Generated traffic. Additional traffic generated by improving the project road, including development traffic.

Existing traffic

The first step in estimating traffic demand is to estimate baseline traffic flows. The Average Annual Daily Traffic (AADT) of traffic currently using the route,

classified into vehicle categories, should be established on the basis of traffic counts. It is normally advisable to carry out a count for seven consecutive days. Counts should be avoided at times when travel activity increases or decreases abnormally. Adjustments should be made for seasonal variations in traffic, either by repeating the seven-day count several times over the year or by using country-wide data indicating seasonal trends.

Vehicle types

Traffic should normally be broken down into at least the following vehicle types:

- cars
- light goods vehicles
- trucks
- buses
- non-motorized vehicles.

Traffic forecasting

Traffic forecasting is a very uncertain process even in a developed economy with stable economic conditions. In a developing economy, the problem becomes more difficult. Since the viability of a road investment in most cases is very sensitive to the forecast traffic levels, the forecasting should be carefully carried out.

A more comprehensive description of traffic forecasting is presented in Chapter 2.

Uncertainty

Both the estimate of baseline traffic flows and the forecast growth rates will be subject to significant uncertainty.

This means that the estimated economic return of the investment will be quite uncertain. It is therefore advisable to evaluate projects using two traffic scenarios, a pessimistic and an optimistic forecast.

Example

Figure 3.1 shows an example of a 15-year traffic forecast for a rural road. The baseline traffic is 290 vehicles per day. No traffic will be diverted, but the road is assumed to generate a traffic volume of 100 vehicles per day in year 1 after the road is improved. Two alternative annual growth rates, 4% and 8%, are used to establish a 'low' and a 'high' forecast.

3.3.2 Vehicle operating cost savings

Vehicle operating costs (VOC) will be reduced when a road is improved. Fuel consumption, wear and tear of tyres, suspension, etc., will be affected when the geometric design is improved and the road surface is made more even. The savings are perceived by the road users in the form of lower expenditures.

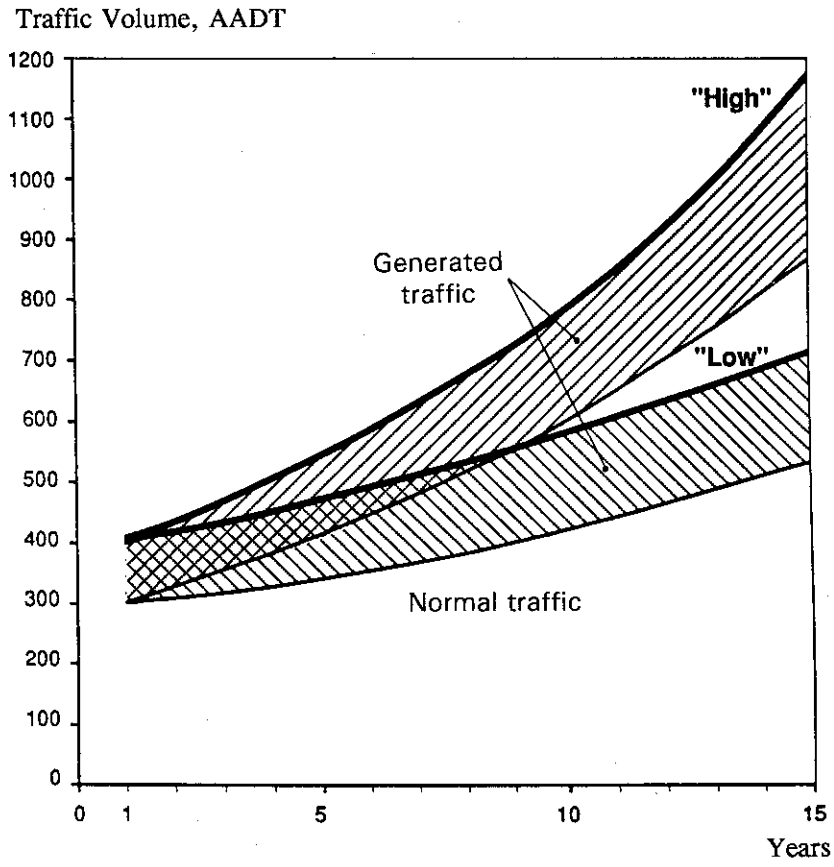
Cost components

VOC consist of the following components:

- fuel consumption;
- lubricating oil consumption;
- spare parts consumption;
- vehicle maintenance labour;
- tyre consumption;
- vehicle depreciation;
- crew costs (trucks, buses, etc.).

Road condition

The World Bank and the TRL have conducted extensive surveys on the relations between vehicle operating costs and road condition (e.g. ref. 2). Tables for estimating operating costs for the various vehicle types can be obtained from



High = 8% annual growth
Low = 4% annual growth

Figure 3.1 Example of traffic forecast.

both the World Bank and TRL. These may serve as tools for manual calculation of vehicle operating costs.

The work, however, has also been computerized in the above-mentioned road investment models RTIM3 and HDM-III.

Whether the calculations are carried out manually or by one of the investment models, the user should provide input data on:

- road characteristics, and
- vehicle characteristics.

Road characteristics

Road characteristics should be measured for the existing road and estimated for the proposed project. Road characteristics include:

- rise ($m\,km^{-1}$);
- fall ($m\,km^{-1}$);
- curvature ($degrees\,km^{-1}$);
- roughness ($m\,km^{-1}$ or $mm\,km^{-1}$);

- road width (m);
- surface moisture contents for unpaved roads (%);
- rut depth (mm).

Roughness is a key factor influencing vehicle operating costs. The measurement of roughness will normally be made with a 'response-type' instrument. In the past, roughness has been measured with a bump integrator (BI). The BI is simulating one wheel of a passenger car. The wheel is mounted via leaf-springs and dash pots in a towed trailer. The unidirectional vertical movement of the wheel with respect to the trailer frame is measured as the device passes over the road at a speed of $32\,km\,h^{-1}$ (20 mph). The displacement is accumulated over a distance of one km and is called the roughness index R with units of $mm\,km^{-1}$. Other measuring methods may be used and the result converted to roughness index.

As an example of a VOC component's dependence on road condition Figure 3.2 indicates the relationship between tyre consumption and road roughness.

VOC savings

The principles of the estimation of vehicle operating cost savings are as follows. For each vehicle type the operating costs are estimated respectively without and with the road improvement. The difference makes up the VOC savings per vehicle using the road.

Total savings to normal traffic are found as the value of normal traffic times the saving per vehicle. Benefits to generated traffic (development benefits) are calculated as half the VOC savings times the estimated volume of generated traffic, if any. See Figure 3.3 for an illustration of the principle.

Where the project results in traffic diversion taking place, all vehicle operating costs on both the road from which the diversion has taken place and on the project road should be considered when determining benefits.

Total vehicle operating cost savings will be calculated for each year of the analysis period. Traffic volumes will grow over the period and VOC savings per vehicle will change according to the way the traffic volume and the road maintenance strategy will effect the road condition.

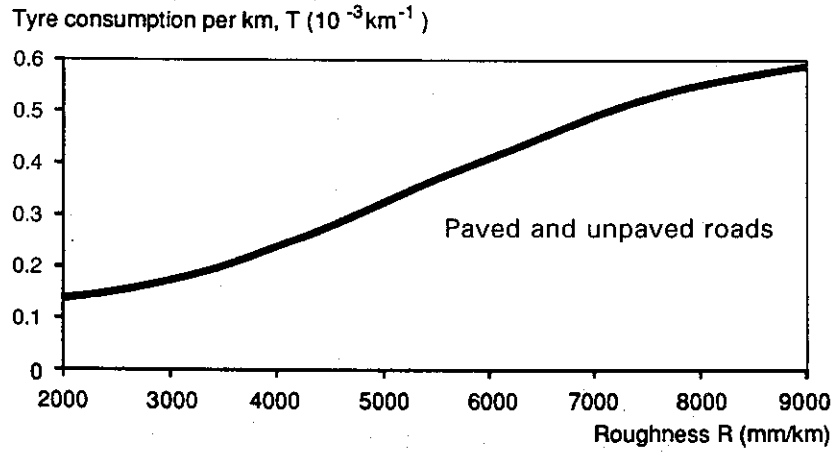


Figure 3.2 Tyre consumption equation for cars and light vehicles (TRRL Laboratory Report 1057, ref. 3).

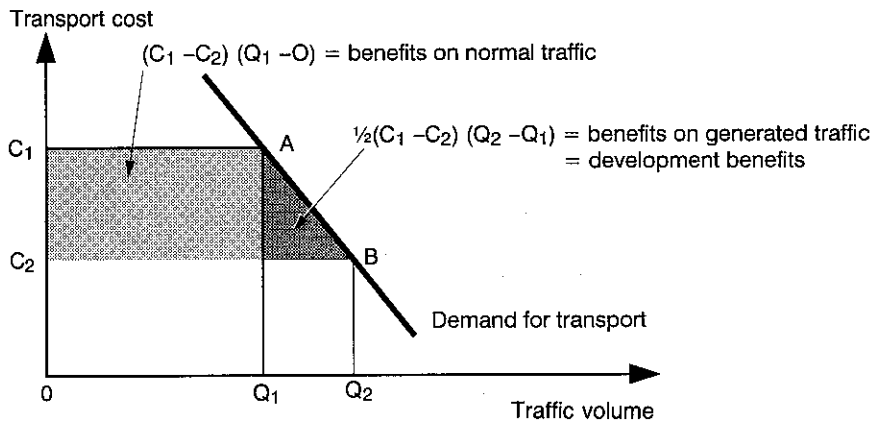


Figure 3.3 Vehicle operating cost savings.

3.3.3 Road maintenance cost savings

With and without project

An economic evaluation should include estimation of the cost of maintaining the road respectively with and without the proposed improvement. In the case of increasing maintenance costs the result is a net cost; in the case of savings the result is a net benefit.

Maintenance savings can normally be expected with the following types of projects:

- paving a gravel road where traffic levels have exceeded the level of economic surface maintenance;
- strengthening or reconstructing a paved road which has deteriorated badly.

If significant traffic diversion from other roads is expected to take place as a result of a new project, then the changing maintenance needs on the road from which the diversion took place should be considered in the estimation of benefits.

Estimating costs

The first step in estimating maintenance costs is to determine a maintenance strategy stipulating maintenance activities in relation to time, traffic and the required service level. Then the activities should be costed.

However, maintenance costing systems that are implemented in organizations are often not accurate enough for determining maintenance cost savings. Real costs are commonly more than 100% greater than those quoted by Roads Departments (ref. 1).

Many costing systems in use only attempt to provide details of total expenditure for budgetary purposes and it is not possible to identify in detail the activities on which expenditures have taken place.

Against this background, it is difficult to obtain realistic unit costs which can be used to determine maintenance savings. However, in most cases, projects will not be justified solely on the grounds of maintenance savings as these will be small in comparison with savings in vehicle operating costs. Nevertheless, maintenance cost estimates are a necessary part of an economic evaluation, including cases where

they are a negative benefit, and an attempt to collect good local cost information must be made.

Implementation

A common weakness of past project evaluations has been to assume a certain maintenance standard and thus certain low vehicle operating costs throughout the analysis period. It has often been experienced that the actual maintenance input was inadequate and the road deteriorated significantly faster than anticipated.

Proper attention must be given in the planning process to establishing suitable maintenance systems for roads to be improved.

3.3.4 Other benefits

In addition to vehicle operating cost savings and road maintenance savings, the benefits of a road investment may include:

- time savings;
- accident savings;
- environmental improvements;
- development benefits.

Time

Shorter road alignments and higher average speeds will lead to savings of time. The benefits of shorter journey times will accrue to passengers being carried and to the commercial vehicle fleet, in that greater vehicle productivity can be achieved.

The time costs of the vehicles used for commercial purposes include the standing costs such as crew wages, vehicle depreciation and interest on capital.

Travel time savings for passengers in buses and occupants of private cars should be divided into time savings during working hours and non-working hours.

In the absence of better data, working hour time can be valued at the average wage rate plus social overheads. The value of non-working time depends on the willingness to pay for the time of those who are commuting or travelling for private purposes. Normally there is no data on this aspect. Non-working hour time is then valued at a certain percentage of working hour time, 0–50%.

In certain rural road evaluations passengers' time savings are valued at zero. When unemployment is significant and wages are low, it is argued that the value of time is insignificant.

Accidents

The number of road accidents can be reduced through the three 'E's: engineering, education and enforcement. Only the engineering factor can be influenced directly by a road project. This factor can be broken down into:

- geometric design;
- road surface;
- road markings and delineations;
- traffic management.

Only a few studies have been carried out in developing countries on the relationships between engineering and accident rates. Therefore estimates of accident reductions must be based on practical experience from the country in question and adjusted research results from the developed world.

It is difficult to attribute monetary values to the losses arising from accidents. In some countries the road authorities have prepared estimates of the average cost to society of road accidents. Reference is made to ref. (1) for a more thorough

discussion of different approaches to the costing of road accidents in developing countries.

Noise and airpollution

It is also difficult to attribute monetary values to losses arising from noise and air pollution and these factors are rarely considered in developing countries.

Development benefits

The extent to which a road investment will generate economic activity in the area adjacent to the road depends on:

- the economic potential in the area, such as unused land
- the relative change in transport costs.

For most road projects where vehicle access already exists development benefits are marginal; user cost savings to normal traffic is the principal benefit. In that case development benefits can be estimated as benefits to generated traffic, as outlined in Section 3.2 and shown in Figure 3.3.

Providing completely new vehicle access or improving a road which is only motorable during the dry season can change transport costs dramatically. This may significantly stimulate development in the area by making it possible to shift from head load or animal transport to vehicle transport. Reduced transport prices will make consumer goods and farm inputs cheaper, and the price at the farm gate of agricultural products will increase.

When large development benefits are expected from a rural feeder road investment it is advisable to adopt the producer surplus approach rather than to estimate the benefits indirectly as benefits to generated traffic. The producer surplus approach is outlined in Section 3.5.

3.4 COST–BENEFIT ANALYSIS

In a cost–benefit analysis estimated costs and benefits over the analysis period are compared in order to ensure that the investment yields a satisfactory economic return.

Investment/ 'do nothing'

Costs and benefits are defined as the difference between, respectively, costs and benefits incurred if no investment was made, and costs and benefits arising as a result of the investment. Thus, a project investment is normally compared to a 'do nothing' case.

However, it is unusual for future investment in such cases to be absolutely zero, as there is normally an existing road or track in existence which in the future will at least require some expenditure or maintenance. If traffic on the existing road is expected to grow rapidly in the future, perhaps because of some complementary investment, then relatively large capital investments may be needed just to prevent the road from becoming impassable. In cases such as this, the 'do minimum' alternative should be considered as the most realistic baseline case against which alternative improvement projects should be evaluated. The choice of an appropriate 'do minimum' case is an extremely difficult decision and has a very large influence on the size of economic return obtained from a project. Considerable attention should therefore be given to its selection.

Costs and benefits for each year of the analysis period should be determined as described in Sections 3.2 and 3.3 above.

Constant prices

All costs and prices should be expressed in constant monetary terms. The effect

of future inflation will be ignored since in most cases it can be assumed to affect costs and benefits equally.

Shadow prices

Economic prices should be used rather than market prices. The purpose is to determine the net economic return to society. Costs and prices should therefore reflect the demand on real resources. This means that tax, duties and other transfer payments must be omitted, and the effect of other market distortions must be adjusted for. The economic prices thus arrived at are termed 'shadow prices'.

For example, the official exchange rate of developing countries often overvalues domestic currency in relation to foreign currency, whereby imported items appear too cheap. On the other hand, official wage rates are often fixed at higher levels than the opportunity cost of labour.

In cases like that, shadow prices for imported goods and local labour should be derived in order to reflect the real scarcity of foreign exchange and the abundant supply of labour.

Discounting

Costs and benefits incurred at different points of time cannot be directly compared, and should therefore be discounted to a base year – normally year 0 of the analysis period. In short, it is more attractive to earn a given benefit today than next year. When a benefit is obtained one year earlier, or a cost is deferred one year, the capital can be invested productively elsewhere in that period and earn an interest equalling the opportunity cost of capital in that country.

The opportunity cost of capital should be used as the discount rate at which all costs and benefits are discounted to the base year. It is no simple task to determine the actual opportunity cost of capital. Therefore, the discount rate to use will normally be provided by the planning authority responsible for the project.

In some industrialized countries the discount rate for public investments is 7%. Several international funding agencies use a rate of 12%. In the absence of specifications by the responsible authority, figures of around 10% are often used.

Future costs (c) and benefits (b) are discounted to present values using the formulas:

$$\text{PCV} = \frac{c_i}{(1 + r/100)^i}; \quad \text{PBV} = \frac{b_i}{(1 + r/100)^i}$$

where:

PCV = present value of cost in year i;
PBV = present value of benefit in year i;
c_i = sum of all benefits in year i;
b_i = sum of all costs in year i;
i = year of analysis where, for the base year, $i = 0$;
r = discount rate expressed as a percentage.

When determining whether an adequate economic return results from making a project investment, the 'net present value' NPV or the 'internal rate of return' IRR decision rules can be applied. In addition, the 'first year rate of return' FYRR rule can be used to evaluate the right timing of the project.

NPV

The NPV is simply the difference between discounted benefits and costs over the analysis period.

$$\text{NPV} = \sum_{i=0}^{n-1} \frac{b_i - c_i}{(1 + (r/100))^i}$$

where:
n = the project analysis period in years;

A positive NPV indicates that the project is economically justified at the given discount rate and, the higher the NPV, the greater will be the benefits from the project. One problem with the use of NPV is that, other things being equal, a large project will have a larger NPV than a smaller one, and on this criterion would always be chosen.

IRR

The IRR is the discount rate at which the present values of costs and benefits are equal; in other words, the NPV is 0. Calculation of IRR is not as straightforward as for NPV and is found by solving the following equation for r:

$$\sum_{i=0}^{n-1} \frac{b_i - c_i}{(1 + (r/100))^i} = 0$$

Solutions are normally found graphically or by iteration. The IRR gives no indication of the size of the costs or benefits of a project, but acts as a guide to the profitability of the investment. The higher the IRR, the better the project. If it is larger than the planning discount rate, then the project is economically justified.

FYRR

The FYRR is simply the present value of the total costs expressed as a percentage of the sum of benefits in the first year of trafficking after project completion. Thus the FYRR is given by:

$$\text{FYRR} = 100 \frac{b_j}{\sum_{i=0}^{j-1} c_i(1 + (r/100))^{j-i}}$$

where j is the first year of benefits, with $j = 0$ in the base year, and other notation is as before.

If the FYRR is greater than the planning discount rate, then the project is timely and should go ahead. If it is less than the discount rate, but the NPV is positive, the start of the project should be deferred and further rates of return should be calculated to define the optimum starting date.

Uncertainties

The results of a cost–benefit analysis are no better than the assumptions and input data on which it is based. The data and parameters used in the analysis of a road project can be prone to substantial errors and it is important to recognize that these exist and to take steps to minimize them.

Scenario analysis should be used for projects that are not well defined. In such cases, a range of scenarios should be examined covering future possibilities that might reasonably be expected to occur. For such scenarios, which will often cover political, economic and social uncertainties, projects should be examined for their robustness in being able to deliver a satisfactory NPV over the range of scenarios considered.

Where projects are well defined, risk analysis is more appropriate and, in these cases, the effect on the NPV of combinations of uncertainties in the project's most sensitive parameters should be examined (ref. 1).

3.5 RURAL ACCESS ROADS

Characteristics

Rural access roads form the last link in the road network connecting the rural areas to the primary roads. They provide access from otherwise isolated agricultural areas to market places, social services, etc.

Rural access roads are normally narrow earth or gravel roads of simple engineering standard and they carry relatively little motorized traffic.

Alternative approach

Since the cost per kilometre of constructing or improving rural access roads is relatively low, a detailed analysis of costs and benefits cannot normally be justified. Moreover, the objectives of improving rural access roads differ from those of trunk road projects. The objectives are often to stimulate agricultural development and improve the living conditions of the local population. Hence, socio-economic development benefits are expected.

For these reasons it will often be necessary to apply evaluation methods that differ from the conventional 'consumer surplus' (road user cost savings) approach outlined in the previous sections.

Cases

Three different cases should be considered:

- a full cost–benefit analysis is required and development benefits are considered significant;
- evaluation of economic return is required but a detailed cost–benefit analysis is not justified;
- evaluation based on non-economic factors is required.

3.5.1 The producer surplus approach

In cases where a full analysis of costs and benefits is required and where the development benefits are expected to be significant, the 'producer surplus' approach is recommended.

When the existing traffic level is low or nil, and the project is expected to generate significant new economic activity, vehicle operating cost savings are both difficult to estimate and a poor expression of project benefits. Normal traffic is insignificant and it is difficult to estimate how much traffic will be generated.

Agricultural surplus

A more direct way of evaluating development benefits is to forecast increases in agricultural production and producers' surplus. Increases in producers' surplus can be predicted from:

- the rise in farm gate prices brought about by the decline in costs of transporting products to market;
- the decline in transport costs of agricultural inputs.

Basic principles

The producers' surplus is simply the value of production, less all costs other than the road itself. The basic principle of the producer surplus method is to value the producers' surplus generated by the road improvement. Normally only agricul-

tural production is considered, as other categories of production are insignificant in rural areas. Thus, the net value added in agriculture is the benefit input to the cost–benefit analysis, while construction costs are the cost input.

Data should be obtained on present and expected future:

- transport costs;
- crop prices at the market place;
- size and crop break down of cultivated area;
- input costs.

In cases where normal traffic and non-agricultural generated traffic are significant, user cost savings to these categories of traffic should be added in the analysis.

Reference is made to ref. (4) for a detailed description of the theoretical framework and practical application of the producers' surplus approach.

Limitations

Unfortunately, the practical application of the agricultural production approach in the field has been poor. The empirical justification for estimating changes in agricultural production has been weak and a failure to consider all the relevant costs of production has often led to the benefits being grossly over-valued. It is recommended to use the approach only when there is a great deal of knowledge about agriculture and its likely supply response to changes in input and output prices (ref. 1).

3.5.2 Simplified economic analysis

A comprehensive economic analysis is a costly and time-consuming undertaking and clearly not something which can be justified for small projects. A simplified approach should then be adopted.

This is the case when for example the task is to determine the economic return and priority rank of a large number of relatively small access road investments.

FYRR

Firstly, it is recommended to use the first year rate of return FYRR as ranking indicator and indicator of economic viability. This will simplify the data collection and forecast of benefits, and it will be sufficiently reliable for already existing roads where traffic growth rates are likely to be similar.

Cost estimation

Estimation of rehabilitation costs should be simplified as necessary. See Section 3.2 for a description of global estimation. Average rehabilitation per km cost figures can be determined for a limited number of typical road and location categories.

Benefit categories

In the case of low-volume-traffic rural access roads, savings in travel time costs and accident costs are likely to be insignificant and can be ignored. The benefits accruing from generated traffic are normally marginal on existing roads which are already open to motorized traffic. For these roads benefits to generated traffic can be excluded as well in a simplified analysis. Thus, vehicle operating cost savings to normal traffic will be the only benefit to consider.

Simplified expression

For rehabilitation projects for existing rural access roads the FYRR can be determined using the following simplified expression (ref. 5):

$$\text{FYRR} = \frac{100 \cdot \text{AADT} \cdot \text{VOCS} \cdot L \cdot 365}{C} \%$$

where:

AADT = average annual daily traffic;
VOCS = estimated vehicle operating cost savings per vehicle kilometre;
L = length of road in km;
C = total rehabilitation cost.

This is still over-complicated for easy application. However, VOCS are directly related to the expected reduction in surface roughness, RR, whence:

$$\text{FYRR} = k\frac{100 \cdot \text{AADT} \cdot \Delta\text{RR} \cdot L \cdot 365}{C}\,\%$$

where k is a constant.

The roughness of the existing road can easily be measured using a bump integrator or roughness metre, or estimated visually by an experienced operator. The roughness of the rehabilitated road can be assessed from the planned design standard.

If the FYRR is required only for ranking purposes there is no need to determine the value of the constant k. However, if the absolute value of FYRR is required in order to indicate the economic viability of each road project, k can be estimated from a conventional estimation of VOCS for a few typical roads among the group of roads.

Development roads

New rural roads opening up isolated areas and rehabilitation projects that change transport costs dramatically can be expected to generate economic development and thus new traffic. Then, benefits of generated activity should be added to the above simplified expression of FYRR, either in the form of benefits to generated traffic or in the form of agricultural production benefits.

The latter can be estimated as gross margins (net farmer surplus per ton) of major cash crops times the expected increase in production of these crops. This requires knowledge of the local agricultural economy.

However, FYRR may not always be an adequate indicator in the case of 'development roads', since it will take some time for the development benefits to materialize. In case the development potential is expected to materialize fully over a 10-year period for all roads considered, it may be appropriate to use the 'fifth year rate of return' (5th YRR) as ranking factors:

$$\text{5th YRR} = \frac{\Delta_5(\text{GM} \cdot P)}{C(1 + r/100)^5}$$

where:

Δ_5 = the difference between the situation in the 5th year and the present situation;
GM = gross margin of major cash crops (monetary units per ton);
P = production of cash crops (ton);
C = construction cost;
r = discount rate (%).

In the case of any VOCS to normal traffic these should be added to the numerator of the above expression.

Simplification

The evaluation of small access road investments can be simplified further by using fairly simple relationships between known parameters (e.g. population and traffic). Thus, estimates of the key parameters used in an economic evaluation may be made without carrying out a full collection of data for each road.

If there has been established a relationship between population in the zone of influence of the roads and traffic levels, or between area of agricultural land in the zone of influence and traffic levels, population and land use data respectively can be used to estimate the traffic levels. If it is further assumed that the savings in VOC per vehicle km are of the same level on all the roads considered, the following economic ranking criteria can be used:

$$\frac{\text{population in zone of influence}}{\text{construction cost}}$$

or

$$\frac{\text{agricultural land in zone of influence}}{\text{construction cost}}$$

The absolute economic return can be found by conducting a conventional calculation of FYRR or IRR for a limited number of the road projects in the programme package.

3.5.3 Ranking based on non-economic factors

Non-economic factors

In some cases the planning organization requires a screening and priority ranking method which takes account of non-economic effects of rural access roads. These effects may include social, environmental and political aspects like for example:

- access to social services (health, education, etc.);
- access to market centres;
- employment generation;
- percentage of small farms in zone of influence;
- impact on soil erosion.

A comprehensive compound ranking method including both economic factors (cost per km, traffic, agricultural production, etc.) and non-economic factors is described in ref. (4).

Principle of ranking

The principles of a compound ranking method are as follows. The factors to be included should reflect the objectives of the road investment programme. First, each factor is measured in its own measurement unit (number of persons getting access to services, tonnes of agricultural output, number of persons employed, etc.) and divided by the length of the road in order to arrive at effect per km figures. The next step is to express all factors on a common scale, often a scale from 0 to 100. Finally political priority weights are assigned to the factors so as to indicate the relative importance of each factor. The projects are then ranked according to the sum of weighted points.

Example

Table 3.1 shows an example of a compound ranking method used in Malaysia (ref. 6). In the example six road projects (A, B, C, D, E and F) are analysed and priority ranked.

Table 3.1 Example of compound ranking method used in Malaysia.

Rural road:	*A*	*B*	*C*	*D*	*E*	*F*	
Kilometres:	*8*	*6*	*7*	*10*	*12*	*5*	*Weight*
Population per km	1100	450	380	290	720	1000	40%
% smallholdings per total hectares	70	80	40	20	60	90	40%
Construction cost per km ($1000)	220	270	210	180	240	300	20%
Adjusted for comparability (0–100), first column: Weighted rating, second column:							
Population per km	(100)40	(41)16	(35)14	(26)11	(65)26	(91)36	
% smallholdings per total hectares	(70)28	(80)32	(40)16	(20)8	(60)24	(90)36	
Construction cost per km	(81)16	(67)14	(86)18	(100)20	(75)15	(60)12	
Weighted rating	84	62	48	39	65	84	
Rank	1	4	5	6	3	1	

First the three factors included are measured in their respective units (the upper half of the table). The achievements are then transformed to a common scale and multiplied by political weights.

Advantages

The advantages of a compound ranking method are that:

- it can be tailored to the specific objectives of the feeder road programme in question;
- it can be simplified and based on input data which is easy to obtain.

Drawbacks

However, the method also has a number of drawbacks that should be considered:

- it is a difficult political and technical task to decide on a common rating scale and political weights;
- there is a risk of double counting if the benefit factors are not properly defined;
- the method does not provide information on the economic rate of return;
- cost and benefit factors are added rather than treated as multipliers.

The last point could be overcome by including only benefit factors in the ranking procedure and then dividing the weighted benefit score by the construction cost in order to find the cost-effectiveness.

Screening

Social and political compound ranking is often used for initial screening of a large group of project candidates. A smaller group of roads thus selected are then priority ranked through an economic evaluation. Used this way the drawbacks of the method are less serious, and it is ensured that only projects fulfilling the broad programme objectives are included in the economic evaluation.

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4 The World Bank HDM model

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4.1 INTRODUCTION

Road investment models

Since the mid-1960s the International Bank for Reconstruction and Development (The World Bank) has been spear-heading the development of methods applicable for evaluation of road plans and setting of priorities for road projects and maintenance in developing countries. A major contributor has been the Transport Research Laboratory (TRL) in the UK who developed the Road Transport Investment Model for Developing Countries (RTIM) (ref. 1). This model was initially based on a comprehensive data collection in Kenya during the 1970s and was, during the 1980s, supplemented with a study in the Caribbean in order to extend the applicability of the Kenya road user cost relationships. The latest edition of the resulting road investment model, RTIM3, was released for commercial use in 1993 (ref. 2).

HDM

By 1966 the World Bank had already developed a preliminary highway design and evaluation model for roads in developing countries (HDM) intended to serve mainly as an engineering tool. The model was based on the AASHO-road tests carried out in Illinois. It became evident, however, that this design guideline was not suitable for reflecting pavement performance in tropical, developing countries. The HDM model was, therefore, modified taking into account the TRL data from Kenya.

Brazil study

In the early 1980s the World Bank lending policy was changed to allow not only financial support for road construction, but also for maintenance. This change of policy led to the need for tools which were capable of evaluating maintenance policies. Consequently, a major research project including a comprehensive data collection was initiated in Brazil, providing further evidence of the nature of road deterioration, road maintenance and road user cost development under tropical conditions (ref. 3). The data collection was concluded in 1984 and the subsequent analysis resulted in important adjustments of the relationships applied in the HDM model particularly concerning maintenance and its effect on road deterioration. This version is labelled the Highway Design and Maintenance Standards Model version 3, HDM-III (refs 4 and 5).

Highway and Traffic Engineering in Developing Countries Edited by Bent Thagesen. Published in 1995 by E&FN Spon, London. ISBN 0 419 20530 6

Overview of HDM-III

The HDM-III consists of a number of mathematical relationships, which forecast the behaviour of a road pavement under specified climatic, maintenance and traffic conditions.

The broad concept of the HDM-III model is quite simple. Three interacting sets of cost relationships are added together over time in discounted present values. Costs are determined by predicting physical quantities of resource consumption and multiplying these by unit costs or prices.

construction costs $= f_1$ [terrain, soils, rainfall, geometric design, pavement design, unit costs];
maintenance costs $= f_2$ [road deterioration (pavement design, climate, time, traffic), maintenance standards, unit costs];
road user costs $= f_3$ [geometric design, road surface condition, vehicle speed, vehicle type, unit costs].

Road user costs include operating costs and time costs. Vehicle speed, which is a major determinant of vehicle operating costs, is related through a complex set of probabilistic functions to road geometric design, surface condition, vehicle type and driver behaviour.

HDM-III applications

The HDM-III is used to make comparative cost estimates and economic evaluations of different policy options ranging from basic maintenance to new construction including different time staging strategies, either for a given road project on a specific alignment or for groups of links on an entire network. It can quickly estimate the total costs for a large number of alternative project designs and policies year by year up to 30 years, discounting the future costs if desired at different postulated interest rates, so that the user can search for the alternative with the lowest discounted total costs.

The use of the HDM-III model requires that appropriate data are available and this will normally require quite extensive surveys.

Limitations of HDM-III

The limitations to the application of HDM-III are mentioned below.

1. The vehicle operating costs submodel has not been validated for congested traffic conditions. That means that the HDM-III is not applicable to urban conditions, and also does not apply on congested interurban highways.
2. The traffic submodel does not endogenously assign traffic within a network. Further, the model does not endogenously redistribute traffic between several routes as a consequence of changes in the relative conditions due to different maintenance and construction alternatives. Such adjustments must be input by the user.
3. The road deterioration submodule has not been validated for freezing climates.
4. The road deterioration model has not been validated for rigid pavements, like Portland cement concrete pavements, and does not include pavement types like penetration macadam. Further, the deterioration models have not been validated for traffic loadings above 1 million equivalent standard axles per year.
5. The model does not endogenously address the issue of road accidents. However, these may be taken into account using the 'exogenous benefit' facility.
6. The model does not include any environmental aspects.
7. The model is not intended for final engineering design, but has been designed

as a tool to assist in the analysis of different alternatives at the project or network level.

8. Bridge analysis is not included in the model.

In spite of these limitations, the present HDM-III is the most comprehensive road works evaluation model available today. Further, considerable work is being carried out in several countries to calibrate the model to local conditions in these countries.

PC applications

The HDM-III has been developed as a computer package available for PCs with 640 kb memory and 20 Mb hard disk. The input module, however, is not very user-friendly and several major users of the HDM-III have established their own interface with the model.

The operations of the Highway Design and Maintenance Standards Model take place in three phases. The first phase is the data input and diagnostics phase. Input data are examined for possible format and numerical errors and internal inconsistencies. Any serious error will produce an error message and stop further execution.

Data requirements

The HDM-III data requirements are:

- inventory and road condition data for existing link;
- new construction data, including cost estimates;
- maintenance options specification, including unit costs;
- vehicle fleet characteristics and operating costs specification;
- traffic data (present and forecast), plus possible generated traffic;
- exogenous development costs and benefits;
- link alternative specification;
- group alternative specification;
- report requests;
- comparison of alternatives report;
- run control.

Traffic scenarios, construction options and maintenance standards are specified independently of the road network. Each scenario, option and standard is given a specific name (ID code) and the appropriate scenarios, options and standards are subsequently related to the specific road link or section through the link alternative specification.

Simulation

The second phase simulates the traffic flows and the changes to the roads as they pass from initial condition through annual cycles of use, deterioration, and maintenance, with possible construction projects to upgrade them. This phase generates information from which it is possible to draw reports for specified periods or annually, giving, for example, road conditions as well as physical quantities and costs for different types of road maintenance and construction works.

The sequence of operations in the simulation phase is shown in Figure 4.1. For each year of the analysis period, the submodels shown are applied in succession to each road link with various alternative construction programmes and maintenance policies that have been specified for it. Each of the submodels is detailed in later sections of this chapter.

Economic analysis

The third phase encompasses economic analysis and comparisons of alternative construction and maintenance policies for selected groups of road links. Reports are generated to produce results in terms of cash flows and economic indicators

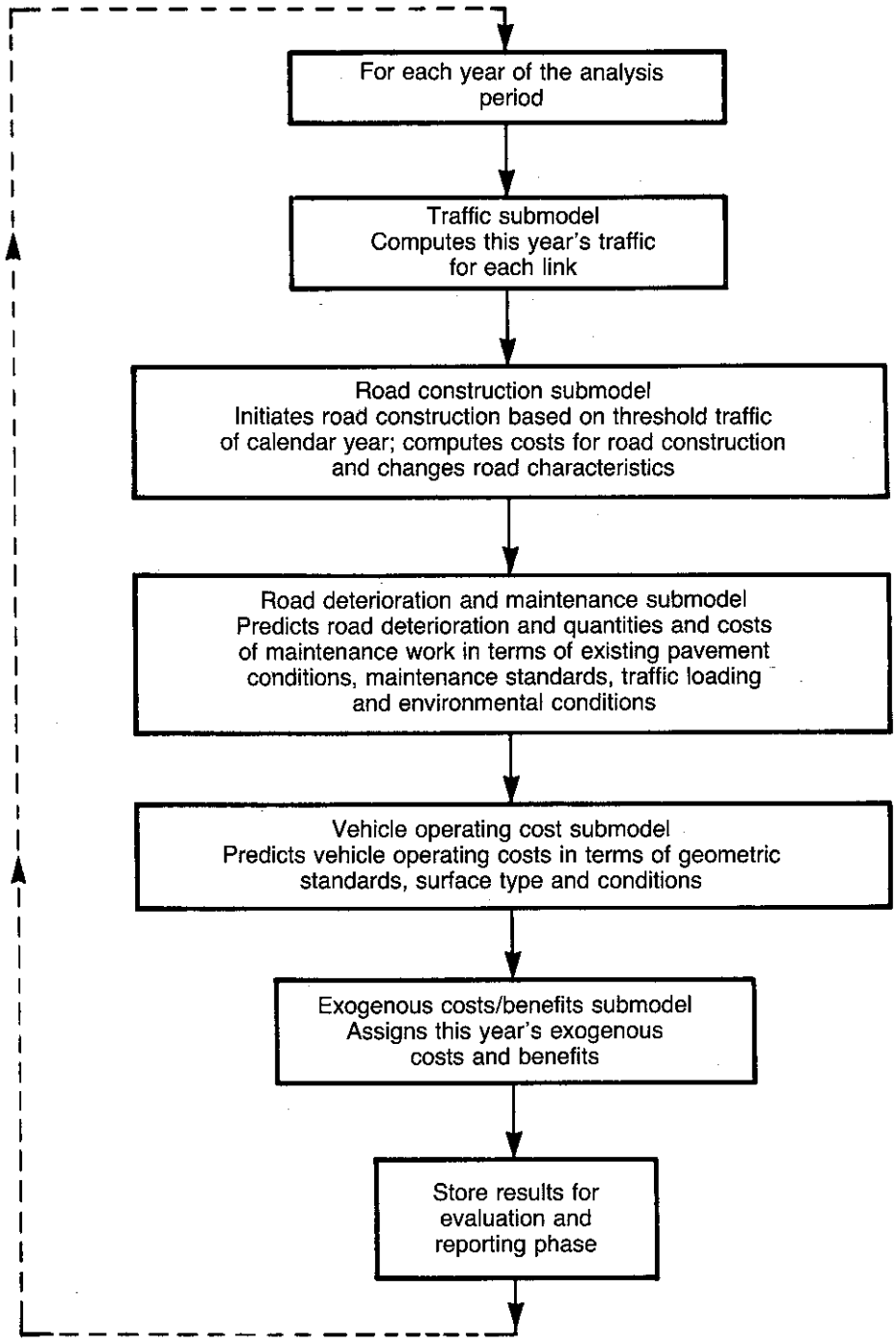


Figure 4.1 Simulation of a link alternative (ref. 4).

like first year benefit, net present value at different discount rates and internal rate of return.

4.2 TRAFFIC SUBMODEL

Specification

The traffic submodel calculates the flow per vehicle group per year and per road link based on user-specified traffic data. The model is able to work with up to nine different vehicle groups.

Normal traffic

For each vehicle group a normal or baseline traffic volume is specified per link. The baseline traffic volume consists of observed traffic and its forecast growth. The user may specify the time profile of flow for a given vehicle group either by specifying flow volumes to take effect in particular years, and remain constant until changed, or by specifying an initial volume and a rate of growth – either a fixed increment or a proportion of the current volume each year. The user may specify different rates of growth and the first year of application.

The time profile of a normal traffic flow is specified to begin in a definite calendar year.

Limitation of submodel

The HDM-III is not able to deal with traffic volumes of less than 1 vehicle per day (vpd) for any vehicle group. This is a limitation which is inconvenient in the Third World. It is not unusual to find that some vehicle groups have less than one vpd. Another problem is that the model cannot handle differing fleet compositions for the existing conditions and the changed conditions due to rehabilitation, construction, etc. It may appear plausible that, for example, replacing a track with a paved road may induce changes in the fleet and thus reduce vehicle operating cost per unit of freight.

Generated traffic

For link alternatives where vehicle operating cost changes due to pavement improvements induce additional traffic (relative to the baseline case) the additional or generated traffic must be specified and added by the model to the normal or baseline traffic. Generated traffic may be specified as a fixed amount of traffic or as a fixed ratio to the normal traffic in the same year. The time profile of the generated traffic is on a relative scale so that it may be initiated by the completion of a construction project, which may be in different years for different alternatives.

Traffic load

Traffic load is an important factor affecting road deterioration, and thus the appropriate maintenance policy. The traffic submodel, therefore, calculates for each link-alternative, the number of vehicle axles and the number of equivalent standard axles going over the road each year.

Traffic report

The HDM-III generates a traffic report per link specified in the set-up of the run. The traffic figures for normal, generated and total traffic per vehicle group as well as equivalent standard axles are presented for each year specified.

4.3 ROAD CONSTRUCTION SUBMODEL

User specification

For the road construction submodel, the user specifies a baseline schedule of construction projects and as many alternative schedules as he wants to analyse. A project may be scheduled to begin in a specific year or may be initiated by the volume of traffic reaching a specified level. The construction period for each project is also specified by the user and may be from one to five years. Projects

may include the construction of new road links or widening, and upgrading of the pavement of existing links.

Procedure

For the baseline schedule and each alternative, the road construction submodel computes the quantities of work and materials required per year and determines their economic and, if required, financial and foreign exchange costs. The submodel modifies the physical characteristics of the road, activates generated traffic and generates any prescribed exogenous costs and benefits in the year after the effective completion.

Construction cost

The total cost per kilometre of a project is made up of various components, and each cost component – except 'overhead' and 'other costs' – is the product of a physical quantity and a unit cost. This applies to:

- right-of-way cost;
- site preparation cost;
- earthworks cost;
- pavement cost;
- drainage cost;
- bridge cost.

The pavement cost may be detailed by layer and drainage cost may be separated into pipe culverts and box culverts. The user may specify, on the basis of separate analyses, all physical quantities per kilometre and all unit costs, as well as 'overhead' and 'other' costs; or, if the multiplications have already been carried out, input may be the resulting cost per kilometre per component or for all components in total, either for an entire link or for separate sections of a link.

Endogenous relationships

Another option makes use of endogenous relationships derived from the empirical studies for estimating the quantities involved in road construction. These built-in relationships compute the quantities as a function of the geometric characteristics, the embankment height and the ground rise plus fall. Combined with the unit costs they provide preliminary cost estimates in case engineering has not been done. The endogenous computation is particularly useful in analysing trade-offs between construction, maintenance, and vehicle operating costs for the analysis of construction standards and maintenance policies at the highway sector level.

4.4 ROAD DETERIORATION AND MAINTENANCE SUBMODEL

General concepts

The road deterioration and maintenance submodel links the construction standards and costs, the road maintenance standards and costs, the traffic and the road user costs. The submodel predicts, for each year, the deterioration of the road surface caused by traffic and climate and the extent to which this is offset by the prescribed maintenance policy. It calculates the maintenance work quantities, and applies unit costs to determine total maintenance cost per year. The physical effects of deterioration and maintenance are simulated on the basis of empirical relations derived mainly from the Brazil study (refs 3 and 6).

4.4.1 Paved road deterioration concepts

Distress modes

Two categories of distress modes are considered for paved roads, that is (a) surface distress as ravelling (loss of surface materials), cracking, and potholing, and (b) deformation distress as rutting and roughness.

Variables The variables to be established at the beginning of each analysis year, either from input or computed, may be summarized as follows:

- road geometry;
- traffic loading;
- pavement structure;
- pavement age and condition;
- environment.

Road geometry The width and the length of carriageway are needed for calculation of total length or area of surface distress, but the geometry has no effect on initiation or progression of different types of distress.

Traffic Two traffic variables are used in the deterioration relationships, the total number of all vehicle axles per lane for the analysis year and the number of equivalent 80 kN standard axle loads per lane based on an axle load equivalency exponent of 4.0. The total number of axles is used for prediction of ravelling and pothole progression and the equivalent standard axles are used in the prediction of cracking, rutting, roughness and indirectly potholing.

Pavement Seven different surface types and three base types are considered.

Surface types:
1. surface treatment (surface dressing);
2. asphalt concrete;
3. slurry seal on surface treatment;
4. reseal on surface treatment;
5. reseal on asphalt concrete;
6. cold mixed asphalt on surface treatment;
7. asphalt concrete overlay or slurry seal on asphalt concrete or asphalt concrete overlay on surface treatment.

Base types:
1. granular;
2. cement stabilized;
3. asphalt.

The surface types 1 and 2 apply to new pavements, while 3 and 6 may be either new or a result of maintenance operations on original pavements. The other types are surfacings after full width maintenance operations on the original pavements.

In the performance prediction model it is necessary to use a measure of pavement strength which summarizes the complex interactions between material types and stiffness, layer thickness, and subgrade stiffness. The strength parameter used in the prediction relationships is the Modified Structural Number (SNC). SNC is defined as follows:

$$\mathrm{SNC} = 0.0394 \sum_{i=1}^{n} a_i h_i + [3.51 \log \mathrm{CBR} - 0.85(\log \mathrm{CBR})^2 - 1.43]$$

where:

a_i = layer coefficient of the ith layer;
h_i = thickness of the ith layer, in mm ($\Sigma h_i < 700$ mm);
n = number of pavement layers;
CBR = California bearing ratio of the subgrade at *in situ* condition (%).

The assumed values of the layer coefficients a_i are presented in ref. (4).

Conversions

Although deflection measurements were generally found to be weak predictors of performance, a correlation was established between strength (SNC) and Benkelman beam deflection (DEF) and the model allows for input of strength parameters as either SNC or DEF or both SNC and DEF. The conversions between modified structural number and Benkelman beam deflection carried out with 80 kN load on the measuring axle were established as follows (ref. 6):

- For pavements with granular base course:

$$\text{DEF} = 6.5\ \text{SNC}^{-1.6};$$

- For pavements with cemented base course:

$$\text{DEF} = 3.5\ \text{SNC}^{-1.6}.$$

Dynamic deflection measurements may be used to estimate the strength parameters, but the user should generate the appropriate conversions to the standard input parameters.

Pavement age and condition

Pavement age and condition (ravelling, cracking, potholing, rutting and roughness) are either provided from input data or computed from the values of the preceding year.

Environment

Environmental factors are incorporated in most of the deterioration relationships through the modified structural number which includes the contribution of subgrade and the layer strength coefficients under *in situ* conditions. Thus, the effects of rainfall and drainage are under normal conditions reflected in these strength variables. Only when, due to special adverse conditions, material properties change significantly with season will it be necessary to estimate different seasonal states.

The effect of rainfall is associated with permeable or cracked surfacing and has been taken into account through the mean monthly precipitation and the cracking parameters in the rutting prediction relationships. The effects of ageing and weathering have been incorporated in the time variables determining the initiation time of surfacing distress for the different surface types.

Deterioration relationships

Paved road deterioration is predicted on a year to year basis through interactive relationships for a number of distress modes and pavement types. The deterioration relationships ultimately manifest themselves in predicted surface roughness, which is the main determinant of the road user costs on a specific length of road.

An important feature of the road deterioration models is the facilities provided for local adaptation of the deterioration relationships through user-specified deterioration factors. These factors are linear multipliers applied to each distress mode in accordance with relevant local experience. The value of the deterioration factors should preferably be established based on quantitative studies of a representative sample of pavements with different strength and age, carrying different traffic volumes.

Surface distress is characterized by an initiation phase and a progression phase whereas deformation distress is continuous, progressive distress which is partly dependent on surface distress. The deterioration prediction relationships for the different distress modes and pavement types control the level of distress or the increment of distress per year of the analysis period. The principles of prediction of primary modes of pavement distress are illustrated in Figure 4.2. The detailed

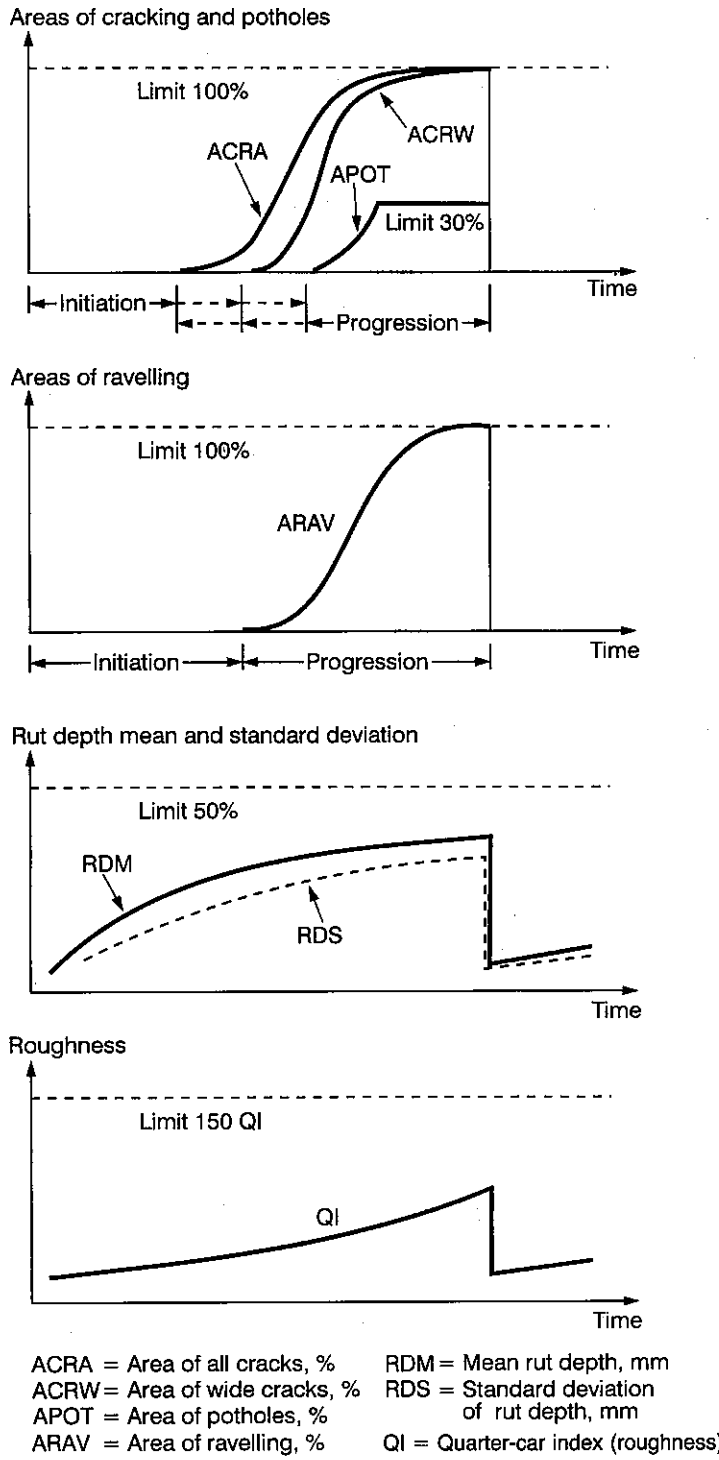


Figure 4.2 Pavement distress prediction (ref. 4).

description of the full set of pavement deterioration relationships will not be given here. Only the roughness progression relationship is presented.

Roughness

The HDM-III model facilitates direct input of roughness in three different scales, International Roughness Index (IRI), Quarter-car index of calibrated Maysmeter (QI) and bump integrator trailer at $32\,km\,h^{-1}$ (BI). Roughness measurements are explained in Chapter 21.

The increment of surface roughness due to road deterioration is predicted for all types of flexible and semi-flexible pavements as a sum of structural deformation related to roughness, surface condition related to changes in cracking, potholing and rut depth variation and an age-environment related roughness term as follows:

$$\Delta QI_d = 13Kgp[134e^{0.023KgeAGE3}(SNCK + 1)^{-5.0}\,YE4 + 0.114(RDS_b - RDS_a) + 0.0066\Delta CRX_d + 0.42\Delta APOT_d] + Kge\ 0.023\ QI_a$$

where:

ΔQI_d = predicted change in road roughness during the analysis year due to road deterioration, in QI;
Kgp = deterioration factor for roughness progression (default = 1);
Kge = deterioration factor for the environment-related annual fractional increase in roughness (default = 1);
AGE3 = age, defined as the time in years since the latest reseal, overlay, pavement reconstruction or new construction;
SNCK = modified structural number adjusted for the effect of cracking, given by:

$$SNCK = \max[1.5;\ SNC - \Delta SNK]$$

where:

SNC = modified structural number;
ΔSNK = predicted reduction in the structural number due to cracking since the last pavement reseal, overlay or reconstruction:

$$\Delta SNK = 0.000\,0758(CRX_a \times HSNEW + ECR \times HSOLD)$$

where:

CRX_a = cracking area weighted for severity of cracking at the beginning of the analysis year (indexed cracking), in per cent of the total carriageway area; maximum value is 63%;
HSNEW = thickness of the most recent surfacing, in mm;
ECR = predicted excess cracking beyond the amount that existed in the old surfacing layers at the time of the last pavement reseal, overlay or reconstruction, given by:

$$ECR = \min[CRX_a - (0.62PCRA + 0.39PCRW);\ 40], \qquad ECR \geqslant 0$$

where:

PCRA = area of all cracking before the latest reseal or overlay, in per cent of the total carriageway area;
PCRW = area of wide cracking (>3 mm) before the latest reseal or overlay, in per cent of the total carriageway area;

HSOLD = total thickness of previous surfacing layers, in mm;
YE4 = number of equivalent standard axle loads for the analysis year, in millions per lane;
RDS_b = standard deviation of rut depth (across both wheel paths) at the end of the analysis year, in mm;
RDS_a = standard deviation of rut depth (across both wheel paths) at the beginning of the analysis year, in mm;
ΔCRX_d = predicted change in cracking index (weighted for cracking severity) due to road deterioration, %;
$\Delta APOT_d$ = predicted change in the total area of potholing during the analysis year due to road deterioration, %;
QI_a = roughness at the beginning of analysis year, in QI.

Guidelines are given in ref. (1) for the determination of the environment-related deterioration factor Kge. For the roughness progression deterioration factor Kgp, as for the factors related to the other distress modes, the most operational way for local adaptation is obviously running the deterioration predictions for a number of well-documented 'pavement cases' in the local environment and establishing the factors as the ratio of observed distress to predicted distress.

An example of the Annual Road Condition Report is shown in Figure 4.3. The example shows an existing gravel road, which after five years is upgraded to a surface dressed paved road.

4.4.2 Paved road maintenance

The HDM model divides maintenance operations for paved roads into two groups and six categories based on when they are applied and their impact on pavement condition and strength.

Recurrent maintenance:
- routine-miscellaneous;
- patching;

Periodic maintenance:
- preventive treatment;
- resealing;
- overlay;
- pavement reconstruction.

Recurrent maintenance

Recurrent maintenance comprises the maintenance operations which are carried out annually, i.e. routine-miscellaneous maintenance and patching. Routine-miscellaneous maintenance is required independent of the traffic volume and includes attention to shoulders, ditches, culverts, slopes and road furniture. The HDM model does not model the effects of alternative levels of routine maintenance. For components that are vital to the pavement structure, i.e. shoulders and drainage features, the model assumes levels of maintenance adequate to assure a normal life for the pavement structure.

The costs of routine maintenance are determined exogenously. The user simply specifies a fixed sum per km per year as the basis for costing routine maintenance.

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ANNUAL ROAD CONDITIONS REPORT

LINK-ALT C012-MN01 SECTION 1

							PAVED ROAD (MEDIUM STRENGTH SUBSECTION)									GRAVEL ROAD	
							END OF YEAR CONDITION										
YEAR	TWO-WAY ADT (VPD)	ANNUAL TWO-WAY ESA (THOUS)	YEARS SINCE LAST SURF/ REGRV	YEARS SINCE LAST CONS	BEF/ AFTER MAINT	SURF TYPE	MOD SN	AREA ALL CRACKS (%)	AREA WIDE CRACKS (%)	AREA RAVLD (%)	AREA POTHD (%)	MEAN RUT DEPTH (MM)	SD RUT DEPTH (MM)	ROUGHNS (IR)	PERIODC MAINT APPLD	GRAVEL THICKNS (MM)	MEAN YEARLY ROUGHNS (IR)
1989	385	13.0	2.		BEF	GRAV										178.4	7.2
					AFT	GRAV										178.4	
1990	413	13.9	3.		BEF	GRAV										156.1	7.5
					AFT	GRAV										156.1	
1991	444	15.0	4.		BEF	GRAV										133.1	7.8
					AFT	GRAV										133.1	
1992	478	16.1	5.		BEF	GRAV										109.3	8.1
					AFT	GRAV										109.3	
1993	514	17.3	6.		BEF	GRAV										84.7	8.4
					AFT	GRAV										84.7	
1994	552	18.6	1.	1.	BEF	ST	2.7	.0	.0	.0	.0	2.2	1.1	2.5	CONS		
					AFT	ST	2.7	.0	.0	.0	.0	2.2	1.1	2.5			
1995	594	20.0	2.	2.	BEF	ST	2.7	.0	.0	.0	.0	2.6	1.1	2.5			
					AFT	ST	2.7	.0	.0	.0	.0	2.6	1.1	2.5			
1996	638	21.5	3.	3.	BEF	ST	2.7	.0	.0	.0	.0	2.8	1.2	2.6			
					AFT	ST	2.7	.0	.0	.0	.0	2.8	1.2	2.6			
1997	686	23.1	4.	4.	BEF	ST	2.7	.0	.0	.0	.0	3.0	1.3	2.7			
					AFT	ST	2.7	.0	.0	.0	.0	3.0	1.3	2.7			
1998	738	24.9	5.	5.	BEF	ST	2.7	.0	.0	.0	.0	3.2	1.3	2.7			
					AFT	ST	2.7	.0	.0	.0	.0	3.2	1.3	2.7			
1999	793	26.7	6.	6.	BEF	ST	2.7	.0	.0	.0	.0	3.4	1.3	2.8			
					AFT	ST	2.7	.0	.0	.0	.0	3.4	1.3	2.8			
2000	825	27.8	7.	7.	BEF	ST	2.7	.0	.0	.0	.0	3.5	1.4	2.9			
					AFT	ST	2.7	.0	.0	.0	.0	3.5	1.4	2.9			
2001	858	28.9	8.	8.	BEF	ST	2.7	.0	.0	.0	.0	3.7	1.4	3.0			
					AFT	ST	2.7	.0	.0	.0	.0	3.7	1.4	3.0			
2002	892	30.1	9.	9.	BEF	ST	2.7	.0	.0	.0	.0	3.8	1.4	3.0			
					AFT	ST	2.7	.0	.0	.0	.0	3.8	1.4	3.0			
2003	928	31.3	10.	10.	BEF	ST	2.7	.0	.0	.2	.0	3.9	1.4	3.1			
					AFT	ST	2.7	.0	.0	.2	.0	3.9	1.4	3.1			
2004	965	32.5	11.	11.	BEF	ST	2.7	.0	.0	8.1	.0	4.0	1.5	3.2			
					AFT	ST	2.7	.0	.0	8.1	.0	4.0	1.5	3.2			
2005	1004	33.8	12.	12.	BEF	ST	2.7	.0	.0	39.5	.0	4.1	1.5	3.3			
					AFT	ST	2.7	.0	.0	39.5	.0	4.1	1.5	3.3			
2006	1044	35.2	13.	13.	BEF	ST	2.7	1.2	.0	82.9	.0	4.2	1.5	3.4	RESE		
					AFT	RSST	2.8	.0	.0	.0	.0	4.2	1.5	3.4			
2007	1085	36.6	1.	14.	BEF	RSST	2.8	.0	.0	.0	.0	4.3	1.5	3.4			
					AFT	RSST	2.8	.0	.0	.0	.0	4.3	1.5	3.4			
2008	1129	38.1	2.	15.	BEF	RSST	2.8	.0	.0	.0	.0	4.4	1.5	3.5			
					AFT	RSST	2.8	.0	.0	.0	.0	4.4	1.5	3.5			
2009	1174	39.6	3.	16.	BEF	RSST	2.8	.0	.0	.0	.0	4.5	1.5	3.6			
					AFT	RSST	2.8	.0	.0	.0	.0	4.5	1.5	3.6			
2010	1221	41.2	4.	17.	BEF	RSST	2.8	.0	.0	.0	.0	4.6	1.6	3.7			
					AFT	RSST	2.8	.0	.0	.0	.0	4.6	1.6	3.7			
2011	1270	42.8	5.	18.	BEF	RSST	2.8	.0	.0	.0	.0	4.6	1.6	3.8			
					AFT	RSST	2.8	.0	.0	.0	.0	4.6	1.6	3.8			
2012	1321	44.5	6.	19.	BEF	RSST	2.8	.0	.0	.0	.0	4.7	1.6	3.9			
					AFT	RSST	2.8	.0	.0	.0	.0	4.7	1.6	3.9			
2013	1374	46.3	7.	20.	BEF	RSST	2.8	.0	.0	.0	.0	4.8	1.6	4.0			
					AFT	RSST	2.8	.0	.0	.0	.0	4.8	1.6	4.0			
2014	1429	48.2	8.	21.	BEF	RSST	2.8	.0	.0	.0	.0	4.9	1.6	4.1	OVER		
					AFT	OVSA	3.6	.0	.0	.0	.0	.7	.2	2.3			

Figure 4.3 Example of Annual Road Condition Report.

Patching

Patching is required depending on the traffic. Patching includes skin patching and slurry seal on cracked or ravelled areas, the replacement of the surfacing in small severely cracked areas, and the filling of potholes. (Slurry seal is a cold mixture of bitumen emulsion and sand applied in a thin layer.) The user specifies patching as either a fixed maximum area per year or as a percentage of the severely damaged area (potholing area). The effect of patching is a decrease of roughness when potholes are patched, but when only cracked areas are treated the effect is normally a slight increase of roughness.

Periodic maintenance

Periodic maintenance is required at intervals of several years depending on the volume of traffic using the road. Selection of the maintenance operation in case more than one periodic maintenance intervention should qualify for implementation in the analysis year is based on the most comprehensive operation taking precedence.

Intervention level

The timing of the periodic maintenance operations may be specified for each type of operation as a scheduled intervention, i.e. fixed intervals or fixed amount per year, or a condition-responsive intervention triggering the intervention when the pavement condition is predicted to reach a user-specified critical threshold level. The user must specify the type of maintenance operation and the physical properties of the operation such as thickness of overlays and the layer coefficients, whereafter the effect of the maintenance on pavement condition, strength and age is calculated endogenously.

Preventive treatment

Preventive treatment is a light sprayed application of bitumen or slurry seal applied on the top of an existing asphalt surface with the purpose of extending the life of the pavement. Under a condition-responsive policy, preventive treatment is performed at the first signs of cracking or ravelling distress. Preventive treatments are not widespread practice and are not discussed later in this book. The effects of preventive treatments are extension of the cracking and ravelling initiation times and decrease of ravelling progression.

Resealing

Resealing comprises surface treatment and slurry seal. A third option is surface treatment with shape correction, an alternative in which some reduction of roughness is achieved through the filling of depressions and repair of damaged areas. Under a condition-responsive policy, a reseal is applied when the unpatched, damaged area or the roughness exceeds a critical level. Following the reseal operation the areas of cracking, ravelling and potholing are set to zero and the deterioration relationships are reinitialized with the new surface type. The pavement strength parameters are updated to take account of the net strengthening due to maintenance and sealing of cracks and the roughness is changed for surface treatment with shape correction.

Overlay

The overlay operation applies to asphalt overlays of less than 125 mm thickness. The asphalt may be hot-mixed asphalt concrete or open-graded cold-mixed asphalt. Thicker asphalt overlays and double overlays, such as granular overlays and cement stabilized overlays with asphalt surfacing, are specified under reconstruction operation. Under a condition-responsive policy, an overlay is applied when the roughness exceeds a critical level. When an overlay is applied the surface type is changed and the new set of relevant surface distress relationships are initialized. The rut depth is reduced by 85% and the roughness is reset to either the value specified by the user for the new surfacing or to the value

computed endogenously on the basis of the overlay thickness and the roughness before overlay is performed. In addition the pavement strength parameters are updated to take account of the net change in pavement strength due to the new overlay and the underlying cracks.

Pavement reconstruction

Pavement reconstruction is a complete rehabilitation of the pavement by asphalt overlays thicker than 125 mm or multiple layer overlays including recycled base and/or surfacing layers. It excludes widening, road realignment, and other geometric reshaping, which should be specified as new construction through the construction submodel. Under a condition-responsive policy, reconstruction is performed when the roughness exceeds a maximum allowable level. When pavement reconstruction is performed, the surface and base types are changed to the new types specified by the user and the appropriate deterioration relationships are initialized. The initial roughness should be input by the user, who should also specify the new pavement strength either as a new structural number or as an increment in structural number due to the pavement reconstruction.

Maintenance costs

The cost of routine maintenance is specified by the user as a fixed sum per km per year while the costs of all other maintenance operations are provided as unit costs per m^2. The cost of periodic maintenance operations should take account of all works involved in the intervention such as road marking, shoulder upgrading, kerbing, adjustment of guardrails, etc., and it should be noted that, since the periodic maintenance costs are computed as the carriageway area times the unit cost per m^2, the costs of such ancillary works must be included as an increment to the unit cost per m^2 of the carriageway. The financial and economic costs of maintenance may be split into separate components of labour, equipment, materials and overhead costs.

Maintenance standards

A maintenance standard comprises a set of maintenance operations with a specified policy of when and under which conditions the operation is to be performed. The maintenance standard may include a number of periodic maintenance operations within each of the maintenance categories, provided the model can unambiguously select the operation to be carried out in a specific analysis year. Similarly multiple maintenance standards may be specified provided that one and only one standard can be selected on the basis of time, surface type and traffic volume specified for the standard.

The optimum maintenance standard for each link, or each of the selected representative links if a whole network is analysed, is identified by running the HDM model with different combinations of maintenance interventions (maintenance standards) and comparing the resulting economic indicators. In Figure 4.4 is shown an example of a tier for evaluation of maintenance standards for a specific road link. The economic indicators are the net present value, the internal rate of return, or the first year benefit. Several model runs are required to complete the evaluation indicated in the tier.

Reports

A breakdown of the total costs by maintenance operation and, if applicable, by labour, equipment, materials and overhead, is given in a Road Maintenance Summary. An Annual Road Maintenance Report presents the quantities involved by maintenance operation by year in the analysis period. Costs for recurrent maintenance are included in 'recurrent maintenance costs' while costs for periodic operations are listed under 'capital maintenance costs'.

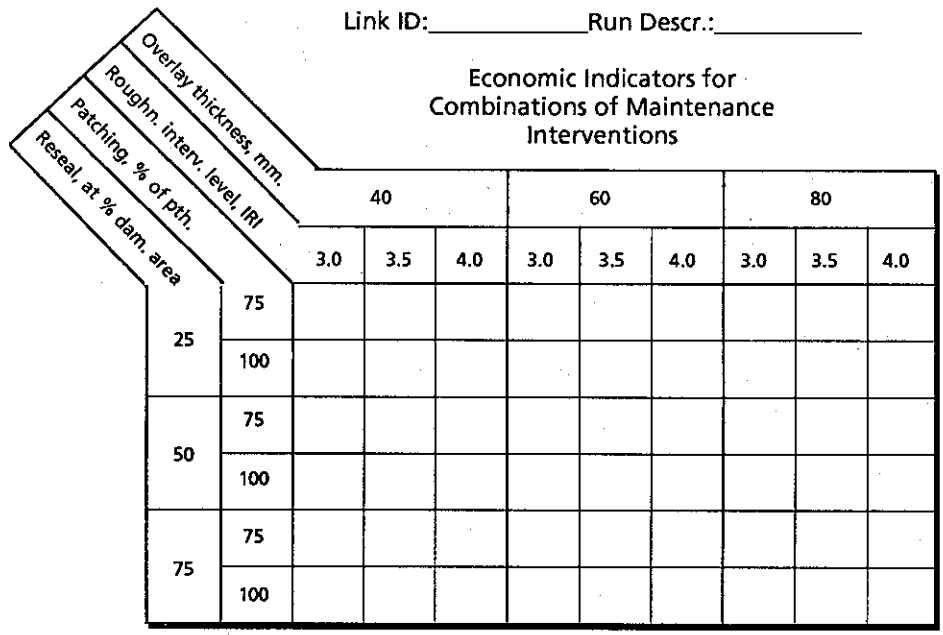


Figure 4.4 Tier for evaluation of maintenance standard.

4.4.3 Unpaved road deterioration

Modelling of the deterioration and maintenance effects for unpaved roads is designed primarily for engineered gravel and earth surfaced roads with moderate to good cross-sectional characteristics with positive crown and reasonable side drains and runoff points.

Distress modes

The deterioration is characterized by roughness and material loss from the surface. The concept of rut depth is not used in the model and is subsumed in the property of roughness.

Variables

The variables in the models are:

- road geometry;
- traffic loading;
- material properties;
- environment;
- grading frequency;
- compaction during construction.

Road geometry

Geometric characteristics are represented by the average horizontal curvature in degrees per km and the sum of the absolute values of vertical rise and fall in m per km which both affect the roughness progression and the maximum roughness. The horizontal curvature affects the rate of traffic-induced material whip-off and the gradient interacts with rainfall in causing material loss by erosion. As for paved roads the width and the length of carriageway are needed for calculation of the total area of surface distress.

Traffic

The traffic loading is the two-way traffic of all vehicles, used in predicting the material loss, and the two-way traffic of light and heavy vehicles, used in the roughness prediction.

Material

The properties of the surfacing material affecting the rate of deterioration are the maximum particle size, the particle size distribution and the plasticity index. From these parameters the minimum and maximum roughness levels are predicted endogenously. However, the prediction should be evaluated against local experience and, if required, the user may specify input values which will override the calculated values. The properties of the subgrade material below the surfacing are not used in the models. The concept of structural number is not used in the models for unpaved roads.

Environment

The environment is represented in the deterioration models only with the mean monthly rainfall, thus making no distinction between uniform and seasonal rainfall climates. It should be noted that poor drainage conditions of the road on level sections have a pronounced effect on the deterioration during high rainfall especially due to rapid development of potholes. Under such conditions the deterioration may be underestimated by the model.

Grading frequency

Grading is the principal maintenance activity for unpaved roads, see the next section. Grading is performed to reduce roughness and restore surfacing gravel from shoulders to the roadway. Grading frequency is the most important parameter in the calculation of the annual average roughness.

Compaction

Unpaved roads may be constructed with full mechanical shaping and compaction or using labour-based shaping and 'natural compaction'. If mechanical compaction is specified in the input, the rate of roughness progression is reduced initially and increased after a few grading cycles.

Deterioration relationship

The deterioration of unpaved roads is predicted through relationships for roughness progression and loss of surfacing material.

Roughness

The relationship for prediction of roughness progression constrains the roughness to a maximum level by a convex function in which the rate of roughness progression decreases linearly with roughness to zero at the maximum level. The maximum level of roughness is a function of material properties and road geometry. The rate of roughness progression is a function of maximum roughness, roughness after last grading, time elapsed since last grading, traffic loading, material properties, and compaction during construction.

Because of the high variability of the variables describing unpaved roads, the roughness prediction standard error tends to be rather large.

Material loss

The loss of surfacing material is a function of traffic loading, material properties, road geometry, and environment. The annual loss is predicted according to the following relationships:

$$MLA = 3.65[3.46 + 0.246MMP \times RF + KT \times ADT]$$

where:

MLA = predicted annual material loss, in mm year^{-1};
MMP = mean monthly precipitation, in m month^{-1};
RF = road rise plus fall, in m km^{-1};
KT = traffic induced whip-off coefficient, expressed as a function of rainfall, road geometry and material characteristics:

$$KT = \max[0; (0.022 + 0.969\frac{C}{57\,300} + 0.00342MMP \times P075 - 0.0092MMP \times PI - 0.101MMP)]$$

C = average horizontal curvature of the road, in degrees km^{-1};
P075 = material passing the 0.075 mm sieve, % by mass;
PI = plasticity index of the material, %;
ADT = average daily traffic in both directions, in vehicles day^{-1}.

4.4.4 Unpaved road maintenance

Maintenance operations for unpaved roads are divided into two groups and four categories.

Recurrent maintenance:
- routine-miscellaneous;
- grading;
- patching.

Periodic maintenance:
- regravelling.

Recurrent maintenance

Recurrent maintenance comprises routine-miscellaneous maintenance, grading, and patching. Routine-miscellaneous maintenance includes maintenance of shoulders, ditches, culverts, slopes, and road furniture. A lump sum cost per km per year is used as the basis for costing routine-miscellaneous maintenance.

Grading

Grading is performed with a motorized or a towed grader. Grading may be specified by the user in one of three ways: scheduled, traffic-responsive or roughness-responsive. Scheduled grading is a fixed number of days between successive gradings. Traffic-responsive grading is a fixed number of vehicle passes between successive gradings. Roughness-responsive grading is a maximum allowable roughness. In all cases, the average roughness between successive gradings is computed as a function of the number of days between gradings. If no grading is specified the long-term average roughness is equal to the maximum roughness.

The effect of grading on roughness depends on the roughness before grading and the material properties. The material properties influence the minimum roughness below which grading cannot reduce the roughness. The repeated cycles of grading and roughness deterioration are treated as continuous by the model. The computed average roughness per analysis year is thus assumed to tend towards a long-term average roughness depending on the traffic volume and grading frequency. Figure 4.5 illustrates the predicted roughness progression and effect of different grading policies for a gravel road with average daily traffic of 300 vehicles per day.

Patching

Patching, also called spot regravelling, is applied to severe depressions, potholes and rutting and may be specified as either a fixed number of cubic metres per km per year or as a percentage of gravel loss to be replaced in the current analysis year. The thickness of the gravel layer is increased and the average roughness is reduced to reflect the volume of material added and the assumption that the gravel is applied in the major depressions and potholes.

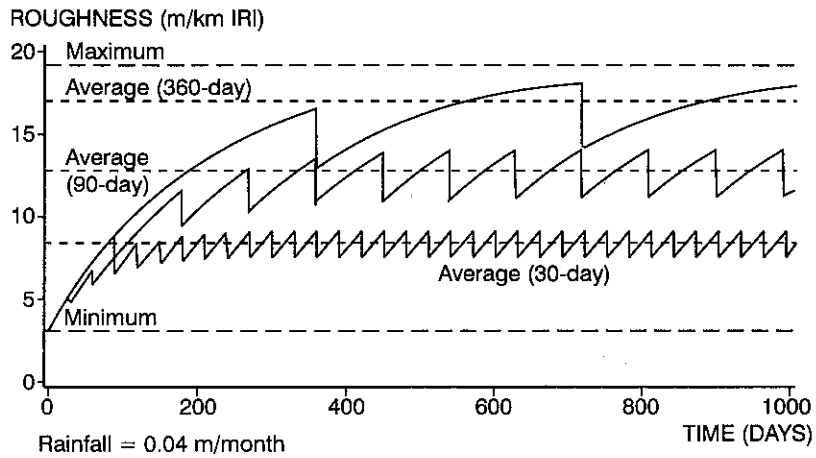


Figure 4.5 Roughness progression and effect of grading (ref. 6).

Regravelling

Regravelling or gravel resurfacing is required at intervals of several years to replace lost surfacing material or increase the thickness of the gravel layer. Regravelling may be specified as scheduled or condition responsive. Scheduled regravelling is applied at fixed time intervals. Condition-responsive regravelling is applied when the current gravel thickness falls below a user-specified minimum allowable thickness.

When regravelling is carried out the gravel material and gravel thickness are changed as specified by the user. The roughness is reset to minimum either as calculated by the model or as specified by the user.

Reports

The reports present breakdowns of quantities and costs for gravel road maintenance operations in a similar way as for paved roads.

4.5 VEHICLE OPERATING COST SUBMODEL

Specification of submodel

The vehicle operating cost (VOC) submodel computes road users' economic costs and, if required, financial costs and foreign exchange costs per road section per year. The quantities of resources consumed and travel time spent are calculated first and then multiplied by prices or unit costs to obtain operating costs and travel time costs. For each representative vehicle type, speeds and resources consumed – fuel, tyres, vehicle maintenance, etc. – are related to the surface type and geometric characteristics of the road section (as initially specified or as altered from time to time by the construction submodel) and to the current roughness of the surface (as determined from the road deterioration and maintenance submodel).

Different VOC studies

The user may choose among four different sets of relationships developed in separate studies in Kenya, the Caribbean, India and Brazil.

The Brazil study

By far the most comprehensive study was carried out in Brazil. Therefore, the Brazil relationships are in general recommended as the basis for the use of the HDM-III. It is stressed, however, that the portability of the Brazilian relationships should be accompanied by an assessment of a number of country-specific

parameters, which for example indicates differences in representative vehicles and speed assessment.

The particular Brazilian relationships apply for speed prediction, fuel consumption, tyre wear, parts consumption and maintenance labour only. The remaining items are common for the four different sets.

The Brazilian relationships are by far the most complicated and it is not possible to evaluate the changes in consumption terms as a result of changes in the road characteristics without intensive use of a computer. Since, however, it is the Brazilian vehicle operating cost submodel which is most widespread the subsequent description is based on this model.

VOC and roughness

The vehicle operating cost submodel estimates the total user costs for a specified maintenance and/or construction alternative. However, in general the HDM-III is used for comparing two specified alternatives in order to determine the more viable alternative. The comparison is based on differences in the cost streams, including differences in the vehicle operating costs. Therefore, it is more important that the vehicle operating costs submodel estimates reasonable differences in the operating costs, for example between smooth and rough pavements, than it estimates total road user costs correctly. And because roughness is the most important parameter for describing the rideability of the pavement and a parameter which is heavily influenced by road works, expressions for evaluating the vehicle operating costs focus on the roughness.

Representative vehicles

The Brazilian model is based on 10 different vehicle types represented by specific vehicles. In particular applications of the model it should be evaluated whether the representative vehicles specified in the Brazilian study can be applied or whether other makes of representative vehicles will have to be specified.

Each of the representative vehicles is described by a number of parameters relating to speed assessment, fuel consumption, tyre consumption, parts and lubricants consumption and maintenance, depreciation and interest, overhead and time costs for goods and passengers.

Further, data concerning axle loads are described as part of the vehicle-specific data. These data are transferred to the traffic submodel.

Speed assessment

Actual vehicle speed on a section is assessed as a function of the road characteristics, the pavement condition and the 'desired speed', that is, the speed a driver would like to apply if the road characteristics and pavement condition were perfect.

Speed is reduced below the desired speed due to variations from the ideal in the parameters listed above. The speed reduction is calculated taking into consideration factors like the driving power and braking power of the vehicle, the gross vehicle weight, the gradient and curvature of the road, and the roughness of the pavement.

The actual speed for a heavy truck as a function of roughness, curvature and gradient is illustrated in Figure 4.6.

Fuel consumption

Fuel consumption is based on the vehicle characteristics, and the nominal engine speed. A large number of parameters relates the fuel consumption to revolutions per minute, maximum used driving power and maximum used braking power. If the fuel consumption model is applied in a country with other representative vehicles than the Brazilian, it is important to validate the parameters.

Tyre consumption

Tyre consumption expressed in the number of equivalent new tyres consumed per 1000 vehicle-kilometres has been evaluated based on an analysis of tyre wear as a function of vehicle characteristics, road geometry and pavement condition.

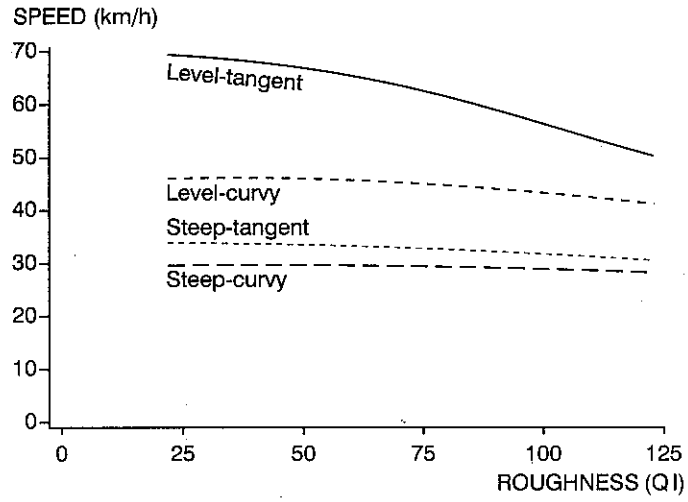


Figure 4.6 Speed as a function of road characteristics for a half-laden heavy truck (ref. 4).

Data concerning cars and utilities were not adequate, and a very simple model based on roughness only has been formulated. The most important parameters affecting the tyre wear for buses and trucks are gradient and roughness. The vehicle weight of the representative vehicle is also of importance.

Tyre wear for a heavy truck as a function of roughness, curvature and gradient is illustrated in Figure 4.7.

Parts consumption

Parts consumption expressed as a fraction per 1000 vehicle-kilometres of the cost of a new vehicle is assessed based on roughness and vehicle average age

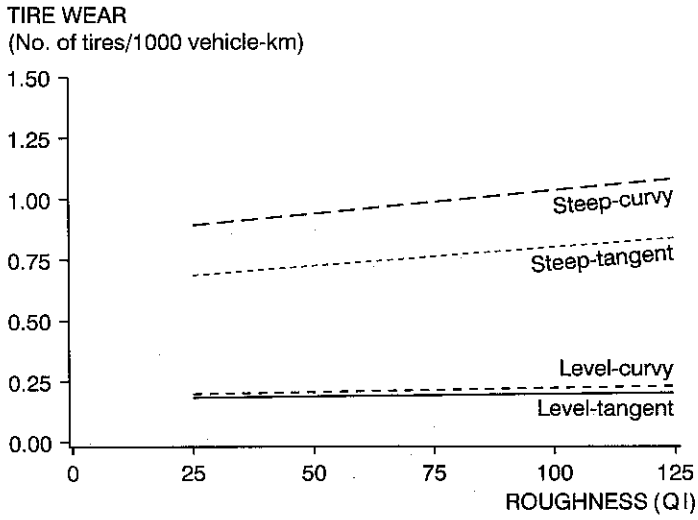


Figure 4.7 Tyre wear as a function of road characteristics for a half-laden heavy truck on a paved road (ref. 4).

expressed as a function of expected lifetime kilometres. The variation with roughness is exponential for small values, and linear for higher values. The point of transition has a major impact on the parts consumption. It is difficult to determine this point based on observations.

Parts consumption can be realistically estimated only within roughness ranges of about 2–10 m km^{-1} measured as IRI.

Maintenance

Maintenance expresses how many hours of labour are used for maintenance work per 1000 km of vehicle-kilometres. Maintenance hours are estimated based on the parts consumption and the roughness.

Lubricants

Consumption of lubricants in litres per 1000 vehicle-kilometres is a function of vehicle type and roughness.

Crew

Cost of crew is considered a variable cost, meaning that crew hours spent on non-driving activities is not included in the HDM-III. Crew hours required per 1000 vehicle-kilometres are inversely proportional to speed.

Depreciation

Depreciation is an accounting term and is rather misleading as a description of this resource cost item. It in fact refers to the impact on average vehicle life, expressed in vehicle-kilometres, of a road improvement and the resultant change in annual cost flows required for fleet renewal. The HDM-III offers the option of two methods for calculating fleet renewal needs expressed as a fraction of the cost of a new vehicle. One method assumes fixed vehicle life, whereas the other method assumes that vehicle service life is related at least in part to the vehicle operating speed.

Average utilization

The average annual utilization expressed in vehicle-kilometres can be calculated by three different methods in the HDM-III:

- The constant annual kilometreage method, assuming a constant kilometreage irrespective of speed improvements. This method is usually applied for non-commercial vehicles.
- The constant hourly utilization method, which assumes that the number of hours driven by the vehicle per year is constant. The method assumes that all speed improvements are converted to benefits.
- The adjusted utilization method, which assumes that only a part of the time saving can be converted to benefits.

The latter is felt to be the most plausible of the three for commercial vehicles, because many freight movement activities are unaffected by the speed of the trip itself – for instance the loading and unloading of the vehicle, refuelling, short-term unemployment between jobs and so on.

Depreciation is calculated based on the fleet renewal requirement which is a function of lifetime kilometres, derived from speed and life in years.

Interest

Interest is also an accounting term and is also misleading as a description. The purpose of this item is to reflect the fact that reduced trip time between two locations permits a given total annual tonnage to be moved by fewer vehicles. Thus, the required fleet can be reduced and the vehicles utilized elsewhere. This cost change in capital immobilized in the fleet is approximated by application of the study discount rate applied to the average value of the vehicle concerned divided by annual vehicle-kilometres. The average value of the vehicle assumes straight-line depreciation and is therefore approximated as being one-half of the acquisition cost.

Overhead

Overhead cost is either assessed as a lump sum per vehicle-kilometre, or as a percentage of the running costs, that is, all costs mentioned above.

Time costs

Passenger and cargo time costs are based on the travel speed and the unit costs per hour per passenger or per ton of goods. Cargo time costs are in general not included. Passenger costs should in general be included. Results are often presented with and without passenger time savings.

Cost items

The vehicle operating costs submodel requires specification of unit cost data for the consumption items. The following unit costs may be specified as economic costs, financial costs and foreign exchange costs:

- cost per litre fuel (petrol and diesel);
- cost per litre lubricant;
- cost per tyre;
- new vehicle price;
- maintenance labour cost per hour;
- crew cost per hour;
- passenger time cost per hour;
- cargo time cost per hour.

Most of these are self-explanatory. Some care should be taken with maintenance labour cost per hour which should take due account of garage overhead costs, buildings, power, tools and so on. Additional attention must, however, be given to ensuring that the output labour cost results are plausible.

Calculation procedure

For each year of each alternative, the model first calculates operating speed for each type of vehicle on each road link. This is dependent on the road geometry and roughness and on the characteristics of the vehicle. From the speed, type of terrain, roughness, and some other factors, fuel and lubricant consumption and tyre wear are determined, as well as parts and labour required for maintenance. For commercial vehicles, crew time on the road is inversely proportional to vehicle speed. For calculating depreciation and interest as proportions of vehicle costs, one is given options of treating vehicle life as constant or varying, and different methods are available for calculating the number of kilometres driven per year and overhead costs. All of these elements, having been calculated in physical or 'real' terms, are converted into monetary, economic, and foreign exchange costs by multiplying them by the user-specified unit costs or prices and, in the case of 'interest', the study discount rate.

Reports

The HDM-III provides a standard report of all vehicle speeds and operating costs per vehicle type for each section specified in the set-up of the run. The report is printed out for the specified number of alternatives and the specified years.

4.6 ECONOMIC EVALUATION AND REPORTING

After all the time streams of physical quantities involved in the baseline and each alternative for each road link have been multiplied by the appropriate unit costs or prices, detailed reports and economic comparisons are prepared. Some reports are standard and are produced automatically; others are selected by the user from a larger number of options. Results may be compared for alternative programmes of

construction and maintenance on individual links. In addition, for comparing possible overall programmes, alternatives for different links are usually bundled together into 'group alternatives'.

The analysis of alternatives in the HDM model can be summarized in the following steps:

1. For each link-alternative, the model separately assembles financial, economic and foreign exchange annual cost streams including the costs of capital investment, recurrent spending, vehicle operation, passenger and cargo time costs, as well as exogenous costs.
2. The annual cost streams for link-alternatives from step 1 above are aggregated for each group-alternative.
3. For each pair-wise comparison of link-alternatives the annual benefit and cost streams are computed for one alternative relative to the other in terms of increases in road capital and recurrent costs, vehicle operating costs and travel time cost savings due to normal traffic, benefits due to generated traffic, exogenous benefits, and total economic benefits. Savings in foreign exchange are also computed.
4. The cost and benefit streams for step 3 above are summarized for each group-alternative comparison.
5. The model then computes for each pair-wise comparison of link-alternatives:
   - the net present value for up to five discount rates as specified by the user and automatically for the zero discount rate;
   - the internal rate of return;
   - the first year benefits.
6. Step 5 is repeated for a pair-wise comparison of group-alternatives.

For sensitivity studies, steps 3–6 above are repeated with certain cost streams multiplied by user-specified factors.

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PART 3

Traffic Characteristics



Heavy traffic in Bangladesh. (Photo by Jorgen Schytte)

5 Highway user behaviour

Jan Kildebogaard, The Danish Road Directorate

5.1 INTRODUCTION

Traffic engineering can in many respects be considered a behavioural science. It is based on the behaviour of the road user in interaction with the physical features of the vehicle and the road. Consequently, a profound knowledge of this interaction is a necessary condition for qualified solutions in road and traffic engineering. Our knowledge, however, is mainly based on European and American experience, but traffic conditions in developing countries quite often differ considerably from this. The economic conditions and the cultural background heavily affect the transport sector and in particular people's attitude towards traffic and safety.

Behaviour

As an example, the capacity of a road may vary substantially depending on the volumes of carts and other slow-moving vehicles. This means that traffic engineering models or measures cannot be applied without a careful analysis of the local conditions, e.g. the variety of vehicles and transport modes may call for a fundamental re-evaluation of the underlying assumptions and recalculation of the parameters.

Capacity

In this context three major areas of particular importance can be identified: the composition of traffic, the behaviour of road users and the condition of vehicles. These three areas will in particular affect the flow of traffic and road safety conditions. In this chapter they are introduced and their influence on the capacity of the road network is discussed, whereas road safety matters will be further dealt with in Chapter 6. A detailed description of the capacity calculation method is found in Chapter 7.

Problem areas

5.2 TRAFFIC COMPOSITION

5.2.1 Problem definition

Traffic in most developing countries is characterized by a variety of transport modes ranging from pedestrians and handcarts, bicycles and rickshaws to trucks

Transport modes

Highway and Traffic Engineering in Developing Countries Edited by Bent Thagesen. Published in 1995 by E&FN Spon, London. ISBN 0 419 20530 6

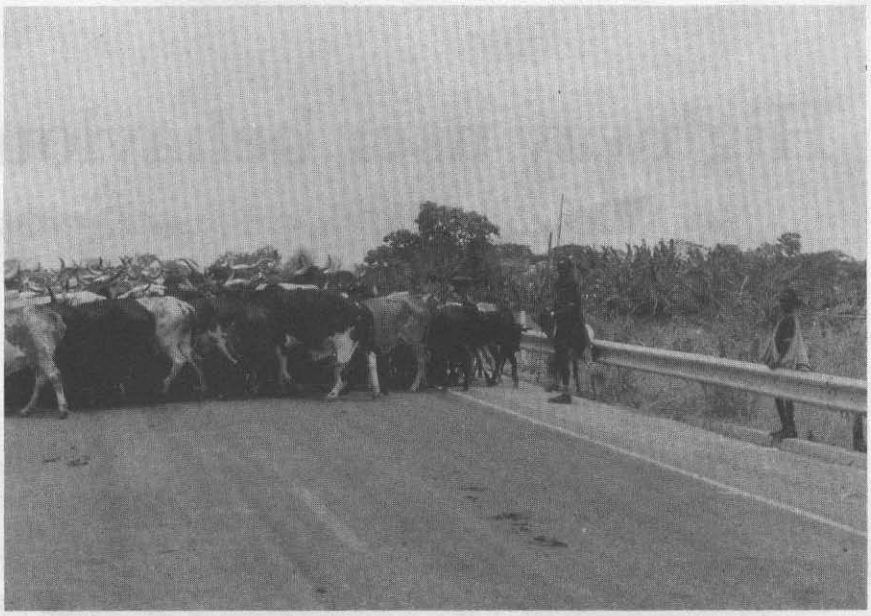


Figure 5.1 The road is often used for driving cattle.

and buses. In addition, other categories of road users, such as cattle (single or in herds) may occupy road space or even block the road completely (Figure 5.1).

Traffic counts

Comprehensive and updated traffic counts will only rarely be available, and the traffic engineer will have either to conduct extensive traffic surveys or to make do with some rough estimates, depending on the actual need. Often the only data available are the number of passenger cars and other motor vehicles. They are usually registered for other purposes such as taxation. In Table 5.1 the distribution of motor vehicles in some countries is shown.

Modal split

The modal split defines the distribution of trips made by individuals using different transport modes, e.g. walking, car, bus or train. To obtain reliable data, comprehensive surveys are necessary because the basic data required are the trips made per person and not per vehicle. Table 5.2 shows figures from some World Bank surveys (ref. 1) and studies in some cities, but unfortunately they do not include pedestrian and bicycle trips.

Table 5.1 The distribution of motor vehicles in some selected countries (data from 1980–83).

Number in 1000	*Egypt*	*Kenya*	*Nigeria*	*Indonesia*	*Thailand*	*Papua New Guinea*	*Philippines*	*Denmark*	*USA*
Cars	580	118	215	866	450	19	342	1440	126 737
Buses	19	6	95	160	67	–	17	8	587
Commercial vehicles	146	80	35	718	433	28	509	245	36 547
Motorcycles	157	17	287	4136	1178	3	218	40	5 584

Table 5.2 Examples of modal share (motorized trips only) in % (ref. 1).

City	*Auto*	*Taxi*	*Bus*	*Paratransit*	*Rail or Subway*	*Other*
Abidjan	33	12	50	–	–	5
Amman	44	11	19	26	0	0
Ankara	23	10	53	9	2	2
Bangkok	25	10	55	10	–	–
Bogotá	14	1	80	0	0	5
Bombay	8	10	34	13	34	–
Buenos Aires	–	–	45	27	–	28
Cairo	15	15	70	–	–	–
Calcutta	–	2	67	14	10	4
Hong Kong	8	13	60	–	19	–
Jakarta	27	–	51	–	1	21
Karachi	3	7	52	18	6	13
Kuala Lumpur	37	–	33	17	0	13
Lima	–	–	45	27	–	28
Manila	16	2	16	59	–	8
Medellin	6	4	85	5	0	–
Mexico City	19	–	51	13	15	2
Nairobi	45	–	31	15	0	9
Rio de Janeiro	24	2	62	2	11	–
San José C.R.	21	2	75	0	0	2
Sao Paulo	32	3	54	–	10	1
Seoul	9	15	68	0	7	0
Singapore	47	–	–	–	–	53
Tunis	24	4	61	–	10	–
London	61	1	23	0	12	2
New York	12	2	14	0	72	0
Paris	56	–	8	0	21	15
Stockholm	48	–	53	–	–	–
Stuttgart	44	6	33	6	–	11
Tokyo	32	–	6	0	61	0
Wellington	56	–	26	–	5	10

5.2.2 Vehicle types

Not only the distribution on transport modes but also the means of transport may differ considerably from European conditions. A wide range of low-cost vehicles has been developed to meet the transport demand, particularly in the rural areas. Table 5.3 shows a list of basic vehicles and their respective performance characteristics in terms of loading capacity, speed, range and relative cost.

Passenger car equivalent (E)

It would be desirable to add to this table the passenger car equivalent (E) for each vehicle. This term indicates how many 'standard' passenger cars the vehicle counts for in traffic. It is used for capacity calculations and will be further discussed in Section 5.7. It must, however, be stressed that it is not a simple matter that easily can be included in the table. E depends on, among other things, the traffic composition and the longitudinal profile of the road.

Table 5.3 Vehicle types and their performance characteristics (ref. 2).

Vehicle	*Load (kg)*	*Speed (km h^{-1})*	*Range (km)*	*Relative cost*
Carrying pole	35	3–5	10	–
Chee-kee: traditional	70	3–5	10	10
improved	70	4–5	10	20
Western wheelbarrow	120	3–5	1	20
Chinese wheelbarrow	180	3–5	3–5	20
Handcart	180	3–5	3–5	30
Bicycle	80	10–15	40	60–100
Bicycle and trailer	150	10–15	40	90–150
Bicycle and sidecar	150	10–15	40	90–150
Tricycle	150–200	10–15	40	150–200
Pack animal	70–150	3–5	20	Varies with species
Animal-drawn sledge (buffalo)	70–150	3–5	20	Varies with species
Animal-drawn cart (oxen)	1000–3000	3–5	50	100–80
Motorized bicycle	100–150	20–30	50	150–200
Motorcycle: 125 cc	150–200	30–60	100	250–600
Motorcycle and sidecar	250–400	30–60	100	350–800
Motorcycle and trailer	200–300	30–60	100	350–800
Motor-tricycle: 125 cc	200–300	30–60	100	500–1000
Single-axle tractor and trailer	1200	10–15	50	1500
Asian utility vehicle	1500	50–80	400	4000

5.2.3 Public transport

Organization

Public transport in developing countries is often a mix between well-organized bus companies (public or para-statal) and private minibus services – more or less organized. The bus companies usually operate a network of fixed routes with fixed timetables and fares. The minibuses may be owned and run privately and individually with a very high degree of flexibility. They may operate like taxis, competing for passengers on the free market. The advantage is that they can easily respond to the actual demand. The disadvantage is, however, that it is very difficult to control their financial circumstances as well as the requirements to drivers and vehicles. This will be further discussed in Sections 5.4 and 5.5.

Distribution

Because of the low proportion of private cars, public transport constitutes the backbone of the transportation system in developing countries, not only in cities but also in rural areas. In Table 5.4 the distribution of trips among various means of public transport is shown in greater detail for some selected cities.

5.2.4 Pedestrians and cyclists

Volume

An outstanding traffic characteristic of developing countries is the large number of pedestrians and cyclists compared to European conditions. As an example, bicycles count for 48% of the total number of trips in Beijing Metropolitan Area (ref. 3). This clearly influences the capacity of the roads as well as the flow of traffic.

Table 5.4 Public transport distribution in selected cities (ref. 2).

	Percentage of trips made by				
City	*Rickshaw*	*Minibus*	*Bus*	*Metro*	*Total*
Calcutta, India	8	4	48	40	100
Delhi, India	17	–	83	–	100
Jakarta, Indonesia	20	–	80	–	100
Chieng Mai, Thailand	7	86	7	–	100
Manila, Philippines	–	64	36	–	100
Surabaya, Indonesia	54	39	7	–	100
Kanpur, India	88	5	7	–	100
Jaipur, India	72	10	18	–	100
Bangkok, Thailand	8	11	81	–	100

Use

In most industrialized countries bicycles are mainly used for individual trips and recreational purposes. In developing countries it is different. Very few people can afford a motor vehicle, and therefore the bicycle serves as a very important means of transport, not only for one person but also for considerable amounts of goods. In addition, the bicycle has been further developed to passenger transport, e.g. the Asian pedicab. The characteristics of some of the bicycle types are shown in Table 5.3.

Walking

The value of a bicycle may equate to several months' salary for a skilled worker, and consequently it may be out of the reach of large parts of the population. This leaves, for a lot of people, walking as the only means of transport.

Problems

Usually the road network and the traffic control measures are designed to meet the demand of motorized traffic in interaction with a limited number of pedestrians and cyclists. Large numbers of pedestrians and cyclists may change the nature of the traffic flow considerably. The capacity of the road sections decreases, and severe problems may arise in the intersections where the secondary conflicts are controlled by give-way regulations. It may be difficult for drivers to find suitable gaps in the slow stream of pedestrians and cyclists, and consequently the formal rules may be violated every so often. An example is the conflict between turning vehicles and pedestrians walking straight on. The only way of overcoming this problem is to design the road network with respect to the actual traffic demand.

5.3 TRAFFIC SURVEYS

Traffic data

As mentioned earlier, reliable traffic data will very rarely be available. Depending on the type of project to be carried out, it may be necessary to conduct traffic surveys or other traffic studies in order to get an updated picture of the transportation patterns, traffic flows or whatever information is needed.

Planning

Before a survey is carried out, comprehensive planning is needed to ensure the greatest benefit of the effort involved. The need for information must be carefully considered and limited with respect to the cost involved to collect the data. Another important issue is to plan for the processing of the data well ahead of the survey. The need for information, the type of survey, the survey forms and the data

processing, analysis and presentation of data must form a coherent whole in order not to waste valuable resources.

Survey types

It is outside the scope of this book to go into detail with the planning and implementation of traffic surveys. A comprehensive description of the topic, including various examples of survey forms, can be found in the TRL Overseas Road Note 11: *Urban Road Traffic Surveys* (ref. 4). Table 5.5 shows a list of different survey types which are treated in the Road Note.

5.4 THE ROAD USERS

5.4.1 Comprehension and attitudes

ness

In many developing countries most of the population has had very little relation to traffic throughout its life. For some (which may be a considerable number) motorized traffic is limited to a bus and some few trucks passing through or near their village every now and then (Figure 5.2). There is no Children's Traffic

Table 5.5 Survey types (TRL Overseas Road Note 11, ref. 4).

Type	*Information*	*Method*	*Output*
Road inventory	Road network characteristics	Observation	Geometry, land-use, road-furniture provision
Parking inventory	Parking supply	Observation	Available parking space, types of parking
Origin–destination	Demand forecasting	Registration number method	Route choice, through-traffic, travel times
Traffic volumes	Demand	Manual counts, automatic counts	Vehicle flows on links, junction movements, passenger flows, traffic variability, peak-hour factors, AADT
Spot speeds	Vehicle performance on links	Short-base method, radar observation	Vehicle speeds on links, speed flow measurements
Network speeds and delays	Route network performance	Floating car method	Network speeds, link speeds, network delay, congestion points
Junction delay	Junction performance	Stopped vehicle count, elevated observer method	Total delays, average arm delays, distribution of delay times by turning movement, delay causes
Saturation flows	Junction capacity	Flow profile method, saturated period count	Saturation flow, junction capacity

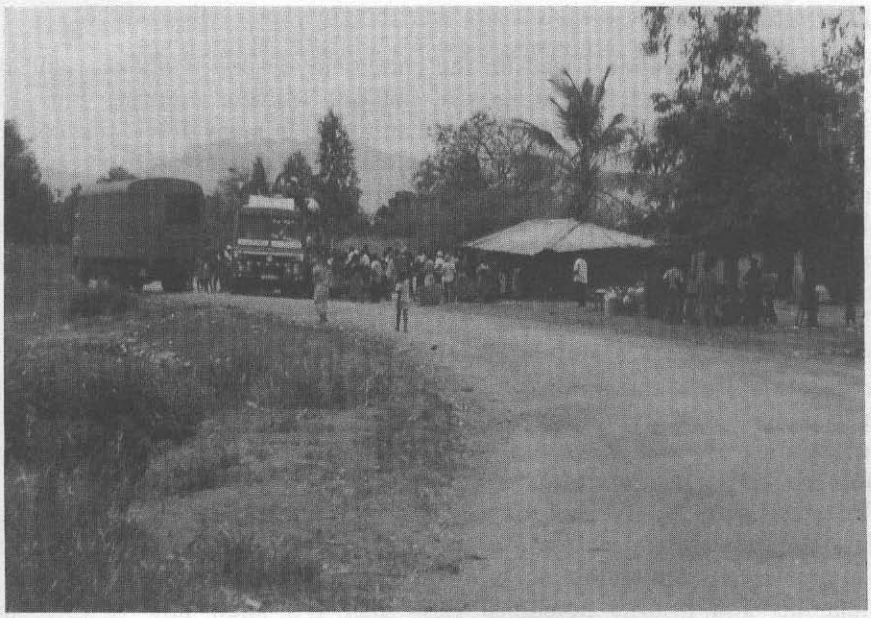


Figure 5.2 Bus stop in the village.

Table 5.6 People who had talked to children about crossing roads (ref. 5).

Percentage of 8–11 year olds in:	*People who had talked to children*						
	Mother	*Father*	*Teacher*	*Policeman*	*Policewoman*	*Friend*	*Other*
Jamaica	63	41	34	28	5	19	7
Pakistan	73	78	37	13	4	16	11
Thailand	53	54	48	7	0	1	1
United Kingdom	95	83	83	64	8	27	57

Organization to create consciousness of traffic and its difficulties among children and hence influence the entire population in some years' time. Table 5.6 (ref. 5) gives an indication of the number of people giving children advice about traffic. It is seen that in all the developing countries the total number of contacts is far below the figures from the UK.

Anticipation

In other words, there is very little common and general understanding of the nature of traffic and its risks. This means that many people are unable to anticipate the consequences of a certain action in traffic, e.g. to estimate the speed of an approaching vehicle before crossing the road or to foresee the track of a turning vehicle. This is very important to keep in mind, because well-intentioned initiatives, such as campaigns and education, may be wasted if they are based on unrealistic assumptions.

5.4.2 Qualifications

Education

Qualified driver education based on the conditions described above would be a very demanding task. It would require not only driving exercises but also a significant adaptation of understanding and attitudes. In many developing countries the only legal requirement is to pass a test combining theory and practice. The driver training can take place in any car, provided that an 'L' (for learner) is displayed on the front and the back of the vehicle, and that a person with a valid driving licence is present. It has even been seen that it is possible to 'buy' a driving licence without a test, although this is hopefully not the common case.

Knowledge

The qualifications of drivers can be assessed in two ways: by asking them questions regarding rules and regulations and the correct way of driving, or by observing their behaviour in traffic. The Transport Research Laboratory has done both in a study covering Pakistan, Thailand and Jamaica (ref. 6). The questionnaires were carried out as a follow-up on behavioural studies showing very little respect for the regulations (Section 5.4.3).

Junctions

The study showed that there were severe gaps in the drivers' knowledge of traffic rules and driving procedures, but only in a few topics was there a widespread lack of knowledge. One of the more serious examples was the questions concerning roundabouts and priority junctions. Less than 1/3 knew how to drive on a roundabout and only 2/3 could answer correctly the questions of how to approach an intersection. In Pakistan, only 50% knew the sequence of the colours in a traffic light.

Other rules

As another example, there was great confusion about the meaning of flashing the headlights although this is mentioned in the Pakistan Highway Code. 17% would flash the headlights to signal to other drivers to drive on, even if the advice of the code is the opposite. In Jamaica, where no advice is given in the Code, 40% would use it in that way. It is obvious that a misinterpretation of the right-of-way may cause conflicts and road accidents.

Another topic regarding the performance of the driver and the vehicle was the drivers' estimate of their stopping distance. Nearly all drivers underestimated the distance at $45\,km\,h^{-1}$ considerably, giving an average value of approximately 5 metres. Only 1/5 answered more than 10 metres whereas the value of the UK guidelines is approximately 23 metres.

Training

The drivers involved in the study were asked how they were trained. Between 19% (Thailand) and 58% (Jamaica) had received professional driving instruction, and between 55% (Pakistan) and 81% (Thailand) had been trained by friends or relatives. 20% of the drivers in Pakistan admitted that they had been driving unaccompanied when they were learning to drive.

5.4.3 Behaviour

Problem

The studies discussed above showed that the drivers had severe gaps in their knowledge. In spite of this, they did better in the tests than should be expected from the previous behavioural studies. The drivers gave, in general, more correct answers than could be expected from their actual behaviour. This was particularly the case with the questions related to pedestrians and their priority. Other studies confirm that the developing countries are faced with a general problem of enforcing even simple traffic rules.

Red signal

Previous TRL studies concerning drivers' reaction at signal-controlled junctions (ref. 7) showed that up to 50% of the drivers observed did not stop at the red light in a junction (Table 5.7).

Table 5.7 Drivers not observing the red light in junction (TRRL Supplementary Report 839, ref. 7).

City		*Number of drivers who had a free choice of stopping or not stopping at red light*	*Number of drivers choosing not to stop at red light*	*Percentage of drivers choosing not to stop at red light*
Ankara (2 sites)	1974	101	36	35
Bangkok (9 sites)	1975	754	391	52
Nairobi (2 sites)	1975	203	101	50
Nairobi (10 sites)	1977	3045	210	7
Surabaya (6 sites)	1975	253	92	36
Surabaya (6 sites)	1976	396	130	48
Central London (11 sites)	1977	364	22	6
Reading Area (19 sites)	1977	726	30	4

Bus drivers

One should expect professional drivers to be better trained and to show a higher capability of driving than other drivers. This does not in general seem to be the case. In many developing countries buses seem to be involved in a relatively high proportion of the accidents. There may be many reasons for this, such as poor bus driver behaviour, inadequate design and maintenance of buses, overloading of buses, poor road user behaviour in general and poor road and traffic conditions. TRL has made a survey of bus driver behaviour in Pakistan (ref. 8) and concluded that the standard of driving was poor. Table 5.8 summarizes the results of the survey. It shows that a very high percentage of drivers made one or more errors performing basic operations in traffic.

In addition to the driving errors mentioned, inappropriate driver behaviour can be experienced in other fields, such as:

- wrong or no light on vehicle;
- overloaded vehicles;
- hazardous overtaking manoeuvres;
- illegal parking;
- illegal loading and unloading of freight and passenger.

5.4.4 Solutions

The examples above show the necessity of improving the overall behaviour of the road users. There are four main areas affecting the driving behaviour:

- driver education and driving test;
- general traffic education including campaigns;
- police enforcement;
- signing and marking.

Table 5.8 Errors made by bus drivers in Pakistan (ref. 8).

Driver error (observation from a following vehicle)	*Percentage of drivers making one or more errors (273 observations)*
STOPPING	
1. Signals omitted or wrong	97
2. Position wrong	95
MOVING OFF	
1. Signals omitted or wrong	99
OVERTAKING	
1. Signals omitted	100
2. Oncoming traffic too near	95
3. Cut in on another vehicle	83
4. Passed too close to a vehicle	51
INTERSECTIONS	
1. Position wrong	64
2. Signals omitted or wrong	45
3. Disobeyed sign or signal	14
4. Speed too fast	2

Education

It is obvious that more emphasis must be put on education and training, not only for drivers but for all potential road users – in other words the entire population. Drivers must be better qualified and driving test requirements should be strengthened. Follow-up tests and training could be considered. As an example, a retraining programme for bus drivers in Pakistan has been introduced based on the poor performance in the survey previously referred to.

Enforcement

Police enforcement is a problem because the police, like most other institutions in Third World countries, lack resources in terms of skilled staff and equipment. Experience from Pakistan shows that the presence of the police may create large reductions in moving violations. Proper enforcement may also require a clearer definition of rules and regulations as well as better information to drivers in terms of signing and marking.

Effect study

Whatever measures are introduced, it is important to carry out a long-term effect study to ensure that the measures actually cause the anticipated results and to adjust the assumptions used for future work.

5.5 VEHICLE CONDITION

5.5.1 The economic reality

The transport sector is particularly sensitive to trade conditions because vehicles, spare parts and fuel are imported articles in most Third World countries. Due to lack of foreign exchange, import is often restricted or prices are immense (or both). Not only spare parts but also skilled mechanics and well-equipped workshops can be scarce resources. Consequently, most of the vehicles suffer from one or more defects.

On the other hand, the transport sector plays a vital role serving other sectors, e.g. agriculture and industry. This means that not only individuals but also society as a whole benefit from keeping vehicles running even if they are far from roadworthy.

5.5.2 Problems

The common problems can be illustrated by figures from a TRL survey on bus conditions in India and Pakistan (ref. 8). Table 5.9 shows the main results which are rather discouraging: practically none of the buses were in working order. Most of them had defective indicators and lights, and 3/4 had faulty tyre pressures or tyres.

Table 5.9 Bus conditions in India (ref. 8).

Vehicle faults		*Percentages of buses with faults (234 buses examined)*
Lights:	rear indicators	99
	rear brake lights	98
	rear lights	92
	rear reflectors	87
	front indicators	97
	main beam headlights	8
	dipped headlights	14
Tyres:	faulty pressure	74
	bald tyre	10
Brakes:	<50% efficiency	31
	play over 0.75 turn	12

In addition to the obvious defects most vehicles will be suffering from a reduced acceleration and braking capability due to age and poor maintenance condition. This will particularly influence the traffic flow parameters, such as starting performance and headway and contribute to a decrease in road and intersection capacity.

5.5.3 Measures

As previously mentioned the feasibility of keeping vehicles in good condition is very limited. Even if a driver is aware of the condition of his car, he may only have the two alternatives of either grounding the vehicle or driving it as it is – and hope for the best. Under such conditions the only effective countermeasures are regular vehicle inspection and intensive police control. Unfortunately, there are severe institutional and practical difficulties involved in both, which will often prevent successful results. Examples are lack of skilled mechanics, vehicles, petrol and equipment and on top of this a widespread corruption due to low salaries and lack of other economic incentives.

5.6 ROADSIDE EQUIPMENT

Information

A precondition for reasonable driver behaviour is that drivers are provided with the information available about routes, road geometry, give-way conditions, etc. This information is usually conveyed by means of road signs, road markings and traffic control devices, such as traffic lights. In addition, street lighting is an important prerequisite for optimum driver behaviour at junctions and on highways with large traffic volumes. Consequently, any road construction or rehabilitation project must include a plan of the roadside equipment.

Traffic signs

Traffic signs include warning signs, regulatory signs and information signs. Warning signs are required to identify actual or potential hazards, e.g. sharp bend, junction, steep hill and road works. The regulatory signs are giving definite instructions to the drivers, such as stop, keep right (left), no entry, no U-turn, etc. Finally, the information signs provide the drivers with information on car parks, filling stations, hotels, etc. and routes (directional signs). Routing information is particularly important, because well-informed drivers will be less distracted and can pay more attention to the traffic situation. Figure 5.3 shows an example of missing signing.

Marking

In addition to traffic signs, road marking is used to provide drivers with warning and information and to emphasize rules and regulations. It consists of longitudinal lines (e.g. lane marking), transverse lines (e.g. stop or give-way lines), kerb marking and words or symbols on the carriageway.

Maintenance

Maintenance of roadside equipment consists of two activities: repair of worn out equipment, and control of the function of the equipment with respect to any changes in the road or traffic conditions. This could be: changing the timing of a



Figure 5.3 Y-junction with no Give Way or route information.

traffic signal controller, changing give-way conditions, changing route information to reflect changes in the road network, etc.

Design speed

The variety of vehicle types and the wide range in speed performance may require a lower design speed than otherwise chosen (see also Chapter 7). This is particularly the case for rural access roads where the possibilities of overtaking a slow moving vehicle are rather limited. The same applies for minor roads through villages where the road serves a number of different purposes.

5.7 CAPACITY

5.7.1 Definitions

Maximum flow

The capacity of a road is defined as the maximum flow of traffic possible under prevailing traffic and road conditions. The capacity is measured as the number of passenger cars per hour. The calculation of the capacity is further described in Chapter 7.

Saturation flow

A fundamental element of the capacity calculation in particular for junctions and road networks is the saturation flow. It is defined as the maximum rate of flow possible in the absence of controlling factors such as the red light in the traffic signal. It can also be considered as the potential capacity of a road section or junction under ideal conditions.

Passenger car equivalent (E)

Normally traffic does not consist of passenger cars only, and consequently other vehicle categories are converted into 'passenger car units' (pcu's) by means of a passenger car equivalent (E). E is defined as the number of passenger cars that are displaced by a single vehicle of a particular type under a prevailing traffic and road condition.

It is worth noting that E is not a fixed value attached to the vehicle type. It depends on two main factors: the road condition, such as the type of road and the gradient (a heavy lorry counts more uphill), and the traffic composition, such as the proportion of heavy vehicles or bicycles. A lot of theoretical and experimental work has been carried out to determine E for different vehicle categories under various road conditions.

Applicability

The previous sections discussed how traffic conditions in most developing countries are considerably different from those in Europe and the United States. They may be different in two neighbouring countries and they may even vary within the country. Consequently, the values of the Highway Capacity Manual (Chapter 7) cannot be transferred without a careful evaluation of local conditions. In many cases they will have to be completely recalculated.

5.7.2 Calculation

Method

The initial method of calculating E was presented back in 1947 by Dr. Greenshields. It is based on measurement of headways between vehicles under saturated flow conditions. This appears for instance when traffic waiting at a stop line starts moving when the traffic signal turns green. The headway of a passenger car following another passenger car is used as the basic value to which other headways are compared.

Examples

In a recent study, Zhao (ref. 3) has used the method to estimate E in mixed traffic with a high proportion of bicycles. A linear regression equation is used to determine E for different vehicle categories. In Beijing, Zhao estimated E for bicycles in mixed traffic to be 0.18. In a pilot study in Copenhagen the corresponding factor was found to be between 0.16 and 0.29, depending on the composition of traffic. E for buses and trucks were found to be between 1.54 and 2.21. In comparison, the city engineer of the Copenhagen Municipality uses the factors 0.2 for bicycles and 2.0 for buses and trucks. The results indicate that the figure of 0.2 may be an adequate estimate for bicycles in most cases.

In an Indonesian study (ref. 9) E was determined using a linear regression equation with several dependent variables, such as the width of the road and the speed of traffic. The results for Jakarta are shown in Table 5.10.

Table 5.10 Passenger car equivalent (E) by road and carriageway type for Jakarta (ref. 9).

	E by road and carriageway type				
	Expressway	*Suburban*		*Urban*	
Vehicle type	*dual*	*dual*	*single*	*dual*	*single*
Car, taxi	1.0	1.0	1.0	1.0	1.0
Truck	1.5	1.5	1.7	1.6	2.3
Small truck	1.0	1.0	1.0	1.0	1.0
Large bus	1.8	2.0	2.6	2.4	3.3
Minibus	1.3	1.4	1.8	1.7	2.6
Opelet	1.0	1.1	1.2	1.2	1.7
Three-wheeled vehicles	–	–	0.8	0.8	0.9
Motorcycle	0.7	0.6	0.5	0.5	0.4
Becak	–	–	0.6	0.6	0.6
Bicycle	–	–	0.5	0.4	0.3

Conclusion

In his study, Zhao concludes that Greenshields' method of determining the headways combined with the regression method calculation of E is applicable to developing countries.

Other methods of calculating E as well as the saturation flow are thoroughly described in ref. (10). Trials have been conducted in Bandung, Indonesia, and three different data collection methods have been tested and compared. It is concluded that all three methods are applicable to Third World conditions.

Traffic models

More complex problems such as the application of a traffic model will often require more detailed analysis. In Santiago, Chile, the TRANSYT computer model to design fixed-time traffic signal plans was introduced. Before the model could be of any use, a comprehensive study was undertaken to determine the basic parameters and to calibrate the model (ref. 11). The main parameters to be analysed were E, the saturation flows and the platoon dispersion parameters, which describe the way a queue of vehicles released from a stop line is dispersed due to differences in acceleration performance and speed.

5.7.3 Means of improvement

One of the most efficient ways of improving the capacity of a road is to separate motor vehicles from slow moving traffic such as pedestrians, bicycles and carts. In Beijing, Zhao (ref. 3) compared the capacity of an intersection approach with mixed traffic flow with a separate bicycle track. Table 5.11 shows that the total capacity of the approach increases when bicycle traffic is separated from motor traffic. These results do not only apply in cities. In rural areas, a combined pedestrian and bicycle path along the road will improve the capacity of the road as well as the safety conditions.

Table 5.11 Comparison of the capacity of an approach with mixed respectively separated traffic in Beijing (ref. 3).

Traffic type	*Motor traffic (car units h^{-1})*	*Bicycles (bikes h^{-1})*	*Total flow (car units h^{-1})*
Mixed traffic	754	4184	1507
Separated traffic	965	4320	1743

Intersections

The above considerations are primarily concerned with road sections and intersection approaches. At intersections special problems arise due to the conflicts between different categories of traffic. As mentioned in Section 5.2.4, large proportions of pedestrians and cyclists may obstruct the intended flow of motorized traffic, and vice versa. The most efficient countermeasure is to design the intersections with a large degree of channelization.

The purpose of the channelization is to separate and direct traffic streams, to provide waiting areas for pedestrians and hence separate traffic conflicts in time and space. It is obtained by means of separate turning lanes and waiting areas, raised traffic islands, pedestrian refuges and carriageway markings. Even a simple pedestrian crossing will benefit from a central island, allowing pedestrians to cross the road in two stages, and at the same time slowing down traffic and preventing overtaking in front of the crossing.

Drivers' task

The overall aim of a wide range of traffic engineering measures, such as channelization, is to facilitate the drivers' task by providing the necessary information and by reducing the conflicts and the choices to be made. In many industrialized countries there is an increasing recognition of the importance of designing the road, its equipment and surroundings to support an appropriate road user behaviour. This may be considered even more important in developing countries where the entire traffic environment including peoples attitude is under development.

5.7.4 Public transport

A large share of trips is made by public transport. Consequently, a considerable improvement in the overall transport system can be achieved by improving the conditions of the public transport. Public transport policy is thoroughly discussed

in ref. (1), whereas this section will deal with the traffic engineering aspects. Similar to the individual traffic, the challenge is to create a traffic system that sustains and facilitates the 'correct' behaviour of the bus drivers as well as the passengers.

Priority

As is the case for non-motorized traffic, segregation of traffic streams can improve traffic flow conditions considerably. The main means are separate bus lanes or even busways. In addition, priority treatment of buses in signal-controlled intersections can improve travel speed. A number of experiments with bus lanes and busways has been carried out in developing countries by the Transport Research Laboratory in the UK. Studies of bus lanes in Bangkok are reported in ref. (12). It is concluded that not only bus travel speed but, more surprising, also car travel speed is improved substantially. One of the reasons is that kerbside parking is banned along the bus lanes.

Busways

A further step is to reserve separate streets or parts of the street as a comprehensive busway network. A recent Research Report from TRL describes the experiences from eight busway case studies in various parts of the world (ref. 13). The general conclusion is that the busways and their facilities have improved traffic operations considerably.

Bus stops

An important issue in relation to public transport is the location and design of bus stops. The stops must be easily accessible for the buses as well as the passengers. The latter should be able to approach the stops without crossing traffic lanes, which often is the case with street cars or bus lanes located in the centre of the road. They must contain waiting areas large enough to accommodate passengers without any danger of being hit by passing vehicles. Larger bus stations must have an easily understood information system to direct the passengers to the appropriate platform. In ref. (12) the layout of bus stops and bus bays is further discussed.

5.7.5 Conclusion

A large number of studies indicates that the traditional traffic engineering calculation methods and models are applicable in developing countries. It is, however, very important to verify the key parameters by updated local traffic studies. Furthermore, the success of introducing new procedures as well as traffic schemes may heavily depend upon the ability of the traffic engineer and his staff to understand and adapt to local conditions.

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6 Traffic safety

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6.1 ACCIDENT DATA IN DIFFERENT COUNTRIES

Police data

In most countries the study of traffic safety is based on the analysis of data on traffic accidents reported by the police.

Divergences

Accident statistics are recorded differently in different countries. Definitions of material damage, personal injury and fatality are not uniform, and even when the definitions are formally uniform, they may be interpreted differently.

Definitions

On the surface a definition like 'fatality' looks fairly simple. However, in order to finalize statistical data it is necessary to set a time limit within which the victim must die in order to be counted as a fatality rather than a serious injury. Some countries use the definition 'died immediately on the scene of the accident', others use a time limit of three days, most countries use 30 days while some use one or two years.

Fatalities

In spite of these problems it is generally assumed that fatality statistics are fairly complete and comparable over time. Also, it is in many countries known how the different definitions compare. As an example: in Malaysia the police have for some years reported both 'died immediately' and 'died within one year'. The last figure is 42% higher than the first one (ref. 1). The corresponding figure in Denmark is not reported but is estimated from health statistics compared with accident statistics to be about the same, probably slightly smaller (35%). Differences in these figures might be due to differences in the rescue system and/or the hospital system.

Motor vehicle statistics which also appear in this chapter are often considered fairly reliable, because they represent data on important taxation items. Also here, however, changes in definitions of types of vehicles frequently occur making time series analysis difficult. Furthermore, for some countries statistical data on accidents and motor vehicles disappear every now and then. In some cases this may be related to wars or disasters but in other cases the reporting seems to have broken down for reasons not stated in the publications. It must be noted that this is a general problem in many developing countries.

Reporting

Statistics are sparse, and statistical reporting is not stable. This is a serious problem for many planning purposes.

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6.2 INTERNATIONAL COMPARISONS OF TRAFFIC SAFETY

6.2.1 Overall comparison between countries

Comparisons in traffic safety whether between road types, road user categories or geographical regions are usually based on the risk concept. In general risk may be defined as: Risk

$$\text{Risk} = \frac{\text{accidental event}}{\text{exposure}} \text{ consequence}$$

In this expression the fraction may be interpreted as a probability: the probability of an accidental event per unit of activity expressed by the exposure. When comparing, for example, road types it seems useful to register accident numbers of a road type in relation to the traffic activity on that road type. So in that case exposure might be vehicle-kilometres driven.

In this context – international comparisons – fatalities are used as the 'accidental event times consequence', i.e. we are considering those cases where accidental events have had fatal consequences. The reason for this choice is stated in Section 6.1.

The next question is which exposure measure to select? Two measures are chosen leading to two risk concepts. Exposure

One risk concept simply measures the number of fatalities in a geographical area in relation to the population in that area. This measure considers traffic accidents as a health problem in line with diseases, other accidents, etc., so we define: Two risk concepts

$$\text{Health risk} = \frac{\text{fatalities}}{\text{population}}$$

The other risk concept attempts to measure accidental events in relation to the traffic activity. One good exposure measure could be motor vehicle-kilometres driven per year in the area. However, this figure is usually not available, so as a substitute measure the number of cars or motor vehicles might be used. The implication is, of course, that motor vehicles are involved in almost all fatal traffic accidents, which is probably true. It is known that the number of kilometres driven per car may differ considerably between countries, so this measure should be applied with care in international comparisons. This risk concept indirectly measures safety in relation to the activity in the traffic system, so we define:

$$\text{Traffic systems risk} = \frac{\text{fatalities}}{\text{motor vehicles}}$$

It is common to define motorization or motor vehicle density of a country as:

$$\text{Motorization} = \frac{\text{motor vehicles}}{\text{population}}$$

It follows directly that:

$$\text{health risk} = (\text{traffic systems risk}) \text{ motorization.}$$

As a first crude application of these risk concepts a number of countries from

Europe, Africa, America and Asia are compared for the year 1989 in Table 6.1 (ref. 1).

The number of cars registered in each country is chosen as the measure of exposure in the traffic systems risk. This is not necessarily the best choice. In some developing countries the number of motor cycles is high so motor cycles might well be included in the exposure measure. However, using cars plus motor cycles instead does not change the overall impression from the table significantly. The major change is that India moves down the scale to a position near Morocco but other changes are of minor importance. For this reason, and in general because of low availability of motor vehicle data, the number of cars is used as the measure of exposure in the following analysis.

The interesting point in connection with this table is that with respect to health risk the ranking of countries is a rather confused mixture of industrialized countries and developing countries. But the ranking according to traffic systems risk is much clearer: on top, typical developing countries are found, in the middle

Table 6.1 Health risk and traffic systems risk for selected countries (1989) listed according to diminishing numerical values (ref. 1).

Country	*Health risk: fatalities per 10^5 inhabitants*	*Country*	*Systems risk: fatalities per 10^4 cars*
South Africa	36.0	Ethiopia	184.6
Swaziland	34.0	India	162.0
Iraq	26.0	Kenya	72.4
Saudi Arabia	23.0	Swaziland	49.6
Taiwan	19.5	Iraq	44.5
France	18.7	Morocco	33.1
USA	18.3	Honduras	30.3
Malaysia	18.2	Yemen	28.3
Poland	17.7	Tunisia	23.7
Canada	16.2	South Africa	22.8
Finland	15.6	Colombia	21.0
Tunisia	14.8	Malaysia	16.7
Denmark	13.1	Taiwan	15.3
Morocco	10.7	Poland	11.4
Israel	10.4	Mexico	5.9
Netherlands	9.8	Saudi Arabia	5.6
Yemen	9.3	Israel	5.0
Great Britain	9.1	France	3.8
Japan	9.0	Denmark	3.5
Norway	9.0	Brazil	3.4
Sweden	9.0	Finland	3.4
Colombia	8.6	Canada	2.6
Kenya	8.6	Netherlands	2.5
Honduras	8.0	USA	2.4
India	6.1	Great Britain	2.3
Mexico	6.1	Japan	2.0
Brazil	3.9	Norway	2.0
Ethiopia	2.2	Sweden	2.0

semi-industrialized countries appear, while at the bottom of the list are highly industrialized countries.

Ethiopia

A peculiar observation is that Ethiopia appears at the bottom of the health risk list, while it appears as number one on the systems risk list. This is possible because motorization is very low. So although the systems risk is very high – about 185 fatalities per 10 000 cars corresponding to almost one person killed in traffic per year for every 55 cars in the country – still the number of cars in the country is so small that there is only about one fatality per 50 000 inhabitants per year, a very small contribution to the overall fatality rate in that country.

USA

Almost the opposite situation occurs, when the USA is considered. The systems risk is very low, but the very high motorization in that country brings the USA almost to the top of the health risk list.

The fact that motorization differs much between countries explains why the apparent 'order' in the traffic systems risk figures disappears in the health risk list.

6.2.2 Problems in international comparisons

There is little doubt that some of the differences in risk figures between countries are due to differences in climate, topography and urbanization. So which are the interests behind international comparisons?

International comparisons may be carried out because of a vague hope that differences in levels might be explained by different safety policies in the countries. If that were the case one country could take over other countries' policies and be sure of benefits.

Cultural differences

Interestingly, this type of study has never produced results which are directly applicable. Useful international comparisons appear through studies of a specific policy in one country, say compulsory wearing of seat belts. The results of such a policy may be transferred to another country under certain conditions. Even when countries seem to be comparable, there may be subtle differences which mean that the results of a policy transfer do not turn out as expected. These differences could be, for example, climatic or cultural. Asogwa (ref. 2) reports that a law in Nigeria requiring the use of crash helmets by motor-cyclists did not work as expected and that this was due to the fact that 40% of those wearing crash helmets did not wear them well strapped at the chin, which means that they are of little use in a collision. The majority of the users thought that strapping was not necessary.

Nordic child cyclist

Another example illustrates that comparisons between nations may cover regional differences which may be the real issue. A study of child cyclist injuries in the Nordic countries gave the result that Denmark had a much higher accident rate than Norway, Sweden and Finland in that order. However, when the data for the countries were broken down into eight climatic regions it appeared that the North-Eastern regions had low injury rates and the South-Western regions had high injury rates. Denmark and Southern Norway were very similar. The difference between Northern Norway and Southern Norway was about the same as the difference between Denmark and Southern Finland. Thus, it appeared that climatic differences could explain part but not all of the differences. Therefore, differences found through comparisons between nations may well be due to differences between regions within countries.

6.2.3 Factors behind risks in developing countries

In spite of the difficulties in international comparisons there is little real doubt that the systems risk understood as the probability of an accident per unit of traffic activity is much higher in developing countries than in industrialized countries.

Basic factors

Traditionally, traffic safety is assumed to be related to the three basic factors in traffic: the road user, the vehicle and the road. These factors will in any country determine traffic conditions such as capacity, speeds, delays and accident risks. It is generally assumed that factors related to road users are of paramount importance to road safety. This must necessarily be so under conditions where vehicles and the road environment are of high quality. But even in countries where vehicles and roads are in bad condition, road user behaviour is essential to safety.

Behaviour

Several studies in developing countries have shown that road users' general level of knowledge concerning motor traffic is not sufficient (ref. 3). Also, the actual behaviour in traffic does not follow the rules. Examples are given in Chapter 5. Another example is given in Table 6.2.

The cases examined in Table 6.2 are such where a pedestrian has stepped into the road and where a driver had a free choice to stop or to continue. The percentage who stopped were recorded.

Table 6.2 Stopping behaviour of drivers at pedestrian crossings (TRRL Supplementary Report 646, ref. 4).

City	*Number of sites studied*	*Percentage of drivers choosing to stop for pedestrians on the crossing*
Bangkok	4	16
Colombo	4	11
Kingston	5	10
Nairobi	6	17
Nicosia	2	17
Surabaya	4	0.2
London 1967	2	73
Reading 1967	1	74
London 1978	5	40
Reading 1978	4	72

It appears clearly that at the sites in England the percentages were the highest and therefore formally best in accordance with international rules. However, from the point of view of actual safety it may be more important to the pedestrians to know what the car driver is going to do. So from that point of view the English driver is in a sense the most unpredictable while the Indonesian driver in Surabaya is predictable but flatly against rules. Only statistical studies of accidents and traffic flows can tell which possibility is, in fact, the safest.

In general it has turned out to be very difficult statistically to prove the safety effects of different types of behaviour. This is also the case with educational programmes. And even proving any safety effects of illegal behaviour – except maybe speeding and drunken driving – is often impossible.

Campaigns

However, it seems obvious in many developing countries that the behaviour of all road user categories – drivers, cyclists and pedestrians – is far from being safe. Therefore, based on common sense rather than statistical studies there is general agreement that behaviour must be corrected. So almost all developing countries conduct some kind of educational or propaganda campaigns, sometimes combined with enforcement programmes. Positive results are reported from Singapore and Egypt (ref. 5).

It is essential in order to achieve positive results from educational campaigns that the message is clear. Zhao points out (ref. 6) that campaign contents in China usually are very general, such as 'safety first, for your and other people's happiness, please observe traffic rules, go out cheerfully go home safely . . .', etc., but there is no evidence that it is useful. On the contrary, effects on behaviour seem to emerge only when the message is clear in terms of telling the road user directly what to do in a given situation.

6.3 TIME SERIES ANALYSIS

6.3.1 Motorization developments

Development stages

Motorization of transport in a country is one indicator of development. In many industrialized countries the number of cars per capita have gone through a development of an S-shaped curve (Figure 6.1).

This development may well be interrupted for a few years now and then due to temporary economic difficulties, etc., but the trend indicated is typical. The three stages of Figure 6.1 could, according to Sicking (ref. 7), be characterized as follows:

Stage I. Developing motorization – economy requires higher priority than safety.
Stage II. Exploding motorization – traffic safety a growing concern.

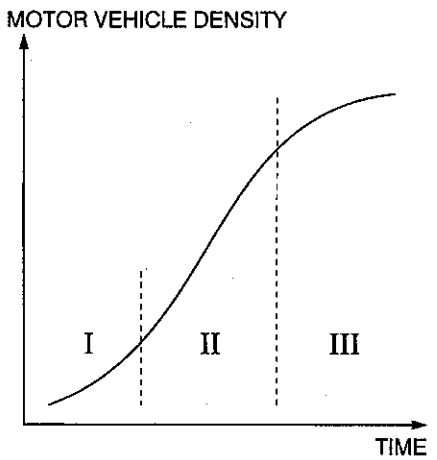


Figure 6.1 Typical curve of the development of motorization in industrialized countries.

Stage III. Saturating motorization – traffic safety to compete with the environment.

Stage I in Figure 6.1 represents early motorization, say, up to 50–100 cars per 1000 inhabitants; stage II will typically go to 300–400 cars per 1000 inhabitants. Developing countries are typically in stage I.

6.3.2 Traffic accident developments

Risk

The following analyses of accident developments are based on fatality data since these data are considered the most reliable. The risk concepts are the same as defined in Section 6.2 and for the reasons given. In most developing countries a measure such as vehicle-kilometres per year is not available. However, it is often assumed that the number of registered motor vehicles is a good measure of traffic activity, since it has been found in industrialized countries that the use of motor vehicles is fairly stable over time. Therefore, a risk based on kilometres driven correlates well with a measure based on registered motor vehicles. So for a given country the developments in a risk figure over time are probably quite reliable, while comparisons between countries of the absolute risk levels should be interpreted carefully. Small differences could easily be without substance.

6.3.3 Smeed's hypothesis

Different relationships between risk and motorization have been hypothesized. Most famous is Smeed's hypothesis (ref. 8):

$$\frac{F}{M} = k \cdot \left(\frac{M}{I}\right)^{-0.67}$$

or

$$F = k \cdot M^{0.33} \cdot I^{0.67}$$

or

$$\frac{F}{I} = k \cdot \left(\frac{M}{I}\right)^{0.33}$$

where:

F = number of fatalities in a given year;
I = number of inhabitants;
M = number of motor vehicles;
k = a constant.

The three formulations in the expressions above are mathematically identical. The first one, being a power function in F/M (= traffic systems risk) and M/I (= motorization), will appear as a linear function in log scales (Figure 6.2).

Motorization

Smeed's hypothesis could be interpreted in two ways. The hypothesis describes national differences in risk levels (traffic systems risk or health risk) between countries as a consequence of different motorization levels. This was the way

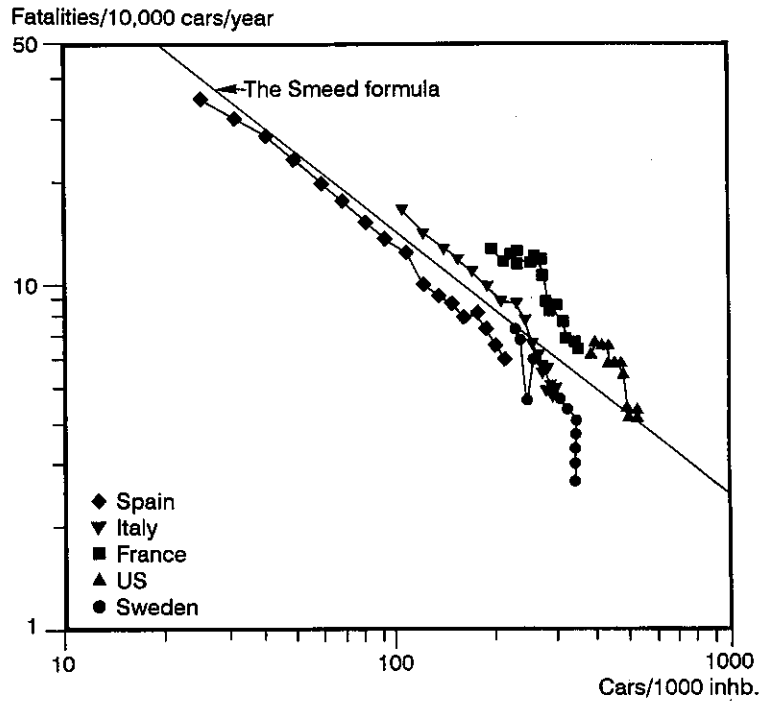


Figure 6.2 Actual development in F/M as a function of M/I for selected industrialized countries (1970–85) compared with Smeed's formula with $k = 3 \times 10^{-4}$.

the hypothesis was first stated, and it was tested in a cross-sectional study of industrialized countries for the year 1938. The relationship was reasonably good. If the relationship holds over time, then the traffic systems risk for a given country (fatalities per 10 000 cars) is simply a function of motorization. Smeed examined this in the case of Great Britain but did not get an equally convincing relationship. Interestingly, the hypothesis may work at lower motorization levels but it does not work at higher levels (Figure 6.2).

The basic idea of the relationship – that fatalities grow with the motor vehicle fleet but not nearly proportional to it – reflects the idea that as motorization grows, traffic safety becomes a growing concern as suggested in phase II of the Sicking model (Figure 6.1), and this concern triggers action which, in turn, reduces the fatality rate. This general view is stated by Jacobs and Hutchinson (ref. 9) who directly point to the necessity of 'higher levels of road user education and training, higher levels of enforcement, better maintenance of vehicles and continuing improvements to the road system' and not just motorization itself if the fatality rate should decrease. It is worth noting that the industrialized countries have worked along the lines mentioned in ref. (9) over several decades.

Social maturing

In generalized terms: higher vehicle ownership (motorization) generates more accidents, but social concern subsequently creates remedial actions in different directions. Remedial actions such as road user education or better maintenance of vehicles are working slowly but may have long-term effects. They could be

considered elements of a social maturing process where society slowly adjusts to the conditions of motorization of road traffic. Therefore, time itself may be a very useful variable in the analysis of traffic systems risk.

6.3.4 An alternative hypothesis

Time as variable

The OECD (ref. 10) suggests that instead of studying fatality rates as a function of motorization, a much simpler relationship emerges when traffic systems risks are studied over time. Figure 6.3 shows plots of traffic systems risk over time for a number of industrialized countries.

The curves, which are approximately linear on a semi-log scale, suggest that the relationship is of the form:

$$\frac{F}{M} = k \cdot e^{((t-T)\cdot \ln a)} = k \cdot a^{(t-T)}$$

where:

F = number of fatalities in year t;
M = number of motor vehicles in year t;
a = annual relative reduction in the fatality rate F/M;
k and T are constants so that if the starting year is T, then k is the fatality rate in year T.

Example

If the systems risk in 1950 was 0.01 and if the risk in any given year is 95% of the risk the year before, then:

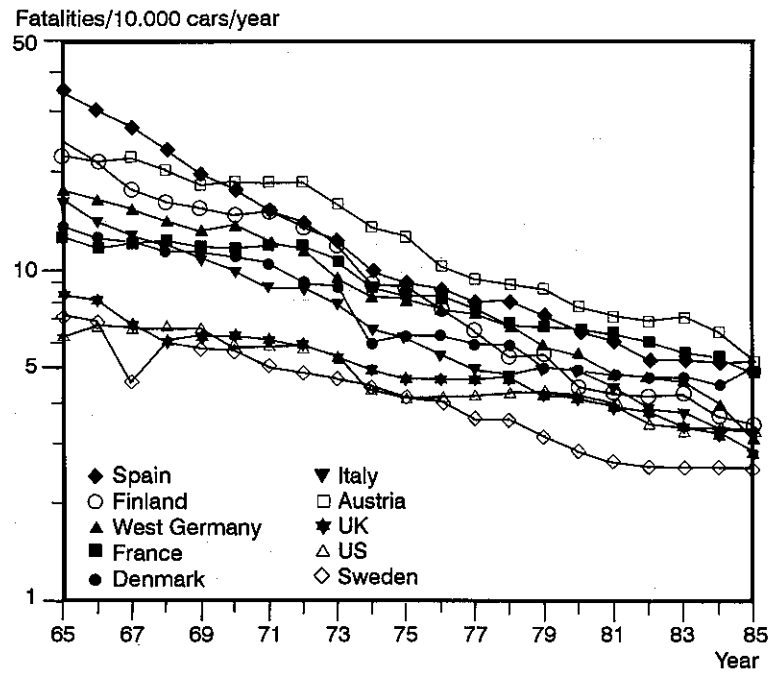


Figure 6.3 Fatality rates for selected industrialized countries, 1970–85.

$$\frac{F}{M} = 0.01 \cdot e^{((t-1950)\cdot \ln(0.95)} = 0.01 \cdot 0.95^{(t-1950)}.$$

According to this model there is an ongoing relative improvement of the traffic systems risk. This does not depend on the actual degree of motorization of a country but rather on the fact that society when started goes on working on the types of safety activities mentioned by Jacobs and Hutchinson. Such policies work over time and do not depend on the degree of motorization.

Figure 6.4 shows a plot of traffic systems risks for a number of developing countries. These plots do not correspond well with the expression for F/M. The risks seem to be more or less constant over time. This means that fatalities grow directly proportional to the number of motor vehicles. This was also true for some industrialized countries (e.g. UK and Denmark) many years ago when motorization was low.

Taken together the observations shown in Figures 6.3 and 6.4 could result in the following hypothesis. Using Sicking's classification it is assumed that:

1. during phase I (developing motorization) the traffic systems risk is constant over time, and
2. during phase II (exploding motorization) the traffic systems risk is an exponentially decreasing function of time.

6.3.5 Consequences of the alternative hypothesis

The perspective for safety improvements in developing countries is apparently not good according to this hypothesis. However, the change from phase I to phase II is

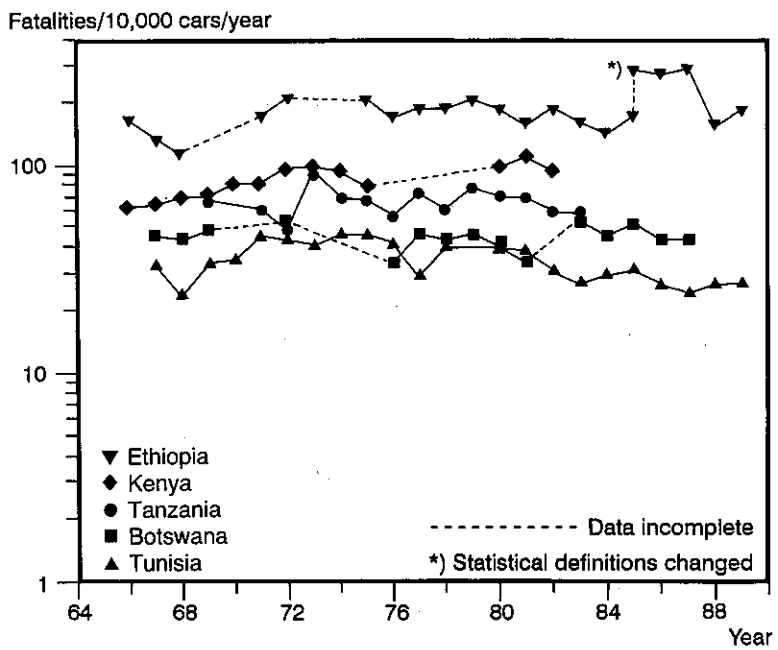


Figure 6.4 Fatality rates for selected African countries, all apparently in phase I.

to some extent political. If it is true that the change has to do with both economic priorities and public concern, there is, in fact, a case for alerting public opinion before death tolls grow too high even during a period when the transport economy is crucial to development. Figure 6.5 shows examples of countries which apparently have changed from phase I to phase II during the 1970s.

Consider the case of China. National accident figures for China are not known. However, Zhao (ref. 6) reports that in 1985 the population of the Beijing district was about 5 million and the number of traffic fatalities about 750 – both figures quite similar to the figures of Denmark in 1985. Given a car density in Denmark about 10 times that of Beijing the traffic systems risk in Beijing is about 10 times that of Denmark.

In spite of China being clearly in phase I there is in China a genuine concern about traffic safety. Appeals for better behaviour in traffic are numerous. Also, horror propaganda appears on billboards in some Chinese cities showing 'last month's traffic victims' in photos taken just after the accidents.

The work on safety is not well organized. Zhao outlines which main activities ought to be undertaken during a national safety programme:

The road user:

- design traffic education courses for primary and high-school students;
- introduce traffic safety clubs for family education of infants;
- develop driving schools. New drivers' training should be regularized and unified.

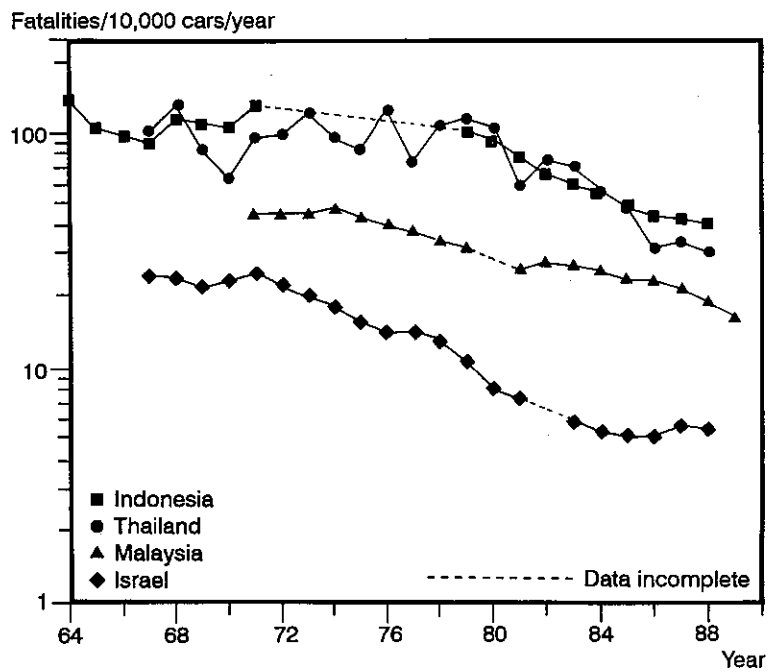


Figure 6.5 Fatality rates for four countries which have changed from phase I to phase II during the 1970s.

The road environment:
- statistical analysis of existing data in order to point out obvious black spots;
- develop road design standards where both traffic operations and traffic safety are taken into account;
- develop uniform standards for road markings.

The vehicle:
- establish standards for vehicle manufacturing and testing;
- develop and apply safety accessories of vehicles, e.g. dualline brake system, seat belts, etc.;
- impose compulsory motor vehicle inspection regulations.

In order to improve the general state of knowledge and create an improved basis for education and research it is recommended to set up in-depth studies of individual accidents.

The outline of safety work suggested by Zhao would have to be adapted to the conditions in a given country. But action plans of that kind may well be a first step towards changing the traffic systems risk of a country from the constant value of phase I into the process of ongoing decrease in the traffic systems risk of phase II. The work is undoubtedly cost-effective considering the fact that even the material damage costs – disregarding personal injuries – are important both in terms of loss of investments and in terms of loss of foreign currency.

6.4 PRACTICAL SAFETY WORK

Accident data

It should be stressed that practical safety work depends critically on access to accident data. It also depends on professionals in the field taking an active role in safety work.

From the traffic and highway engineering point of view an interesting question is to what extent experience from Western countries can be transferred to developing countries? In organizing local safety work there are two main lines of approach to be considered:

- develop a local accident registration system; perform the necessary analysis in order to identify main local problems and develop the necessary action programme;
- apply Western engineering experience in terms of planning methods, design standards, road markings, methods of accident analysis, etc.

The development of local accident registrations and local problem identification is necessary in the long run in order to achieve maximum efficiency of the safety work. Only in that process may local insight into the cultural, social and geographical circumstances be fully utilized. However, in most developing countries it is not advisable to postpone action until this process is well under way because motorization and traffic accidents are increasing rapidly.

Therefore, it is necessary also to utilize experience from industrialized countries. An example of how experience might be presented and summarized is given in the book: *Towards Safer Roads in Developing Countries* (ref. 11). The book presents

chapters on planning, design, traffic operations and accident countermeasures. Many examples illustrate problems and solutions.

Accident countermeasures

Most industrialized countries experienced during phase II – the exploding motorization – that the road network developed high-risk locations, so-called 'black spots'. Such locations may have been risky all the time, but when accidents became numerous, the risks became visible and locations could be identified through statistical analysis. Many countries have implemented very successful black spot programmes over the years. Black spots often represent sudden changes in the road standard along the road such as very sharp bends or very short sight distances which may be a surprise to drivers. Many such difficulties may be overcome by low-cost countermeasures such as good road markings, removal of vegetation at junctions etc.

In ref. (11) there are many examples of problems identified in developing countries but most examples of countermeasures come from developed countries. It seems that in spite of the positive Western experience black spot action programmes are not given high priority by many officials in developing countries – an impression which finds some support at international meetings. There may be good reasons for this, such as lack of data, lack of identification methods, lack of experienced traffic engineers, low maintenance budgets, etc. However, the real reason may well be that it is politically more significant to create new infrastructure than to eliminate accidents. So officials may be tempted to allocate all available resources to new investments and disregard operations and maintenance – and therefore indirectly also safety – of the existing road network.

6.5 GENERAL OUTLOOK

The analysis of accident data from developing countries suggests that accidents are likely to develop in proportion to the motor vehicle development up to fairly high degrees of motorization. Halting this development requires very determined action by political or governmental officials.

If it is possible to change the fatality rate from being constant into the exponential decrease at an early stage, quite substantial savings are possible both in terms of human suffering and of economic losses. Some donor agencies have started financing road safety programmes.

High priority should to be given to registration systems for road accidents. Accident data are in any case the indispensable foundation for a road safety programme.

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PART 4

Geometric Design



Landslide blocking a winding road in Bhutan. (Photo by Mark Edwards/Still Pictures)

Geometric design controls

7

Kent Falck-Jensen, Cowiconsult

7.1 APPROPRIATE GEOMETRIC STANDARDS

Traffic requirements

Geometric design is the process whereby the layout of the road in the terrain is designed to meet the needs of the road users. The needs of road users in developing countries are often very different from those in the industrialized countries. In developing countries, pedestrians, animal-drawn carts, etc., are often important components of the traffic mix, even on major roads. Lorries and buses often represent the largest proportion of the motorized traffic, while traffic composition in the industrialized countries is dominated by the passenger car. As a result, there may be less need for high-speed roads in developing countries and it will often be more appropriate to provide wide and strong shoulders. Traffic volumes on most rural roads in developing countries are also relatively low. Thus, providing a road with high geometric standards may not be economic, since transport cost savings may not offset construction costs. The requirements for wide carriageways, flat gradients and full overtaking sight distance may therefore be inappropriate. Also, in countries with weak economies, design levels of comfort used in industrialized countries may well be a luxury that cannot be afforded.

Road safety

Little research has been carried out in developing countries on the relationships between accident rates and road geometry, but that which has been undertaken indicates that the number of junctions per kilometre appears to be the most significant factor, followed by horizontal and vertical curvature.

High accident rates have been observed on gravel roads. Among the possible causes of this might be poor geometry, slipperiness of the surface in wet weather and poor visibility caused by dust and high vehicle speeds. Some studies have shown that accident rates decrease with reduced road roughness and it is likely that by keeping gravel road surfaces well maintained or by paving gravel roads, accident rates will be reduced.

Results so far obtained suggest that the accident rates in developing countries are considerably higher than in developed countries for similar levels of vehicle flow and geometric design. This is probably because of factors such as road user behaviour, vehicle condition and maintenance. Thus, from the point of view of

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safety, it appears that geometric standards used in the developed countries are not fully applicable to the developing world.

Development levels

When developing appropriate geometric design standards for a particular road in a developing country, the first step should normally be to identify the objective of the road project. It is convenient to define the objective in terms of three distinct stages of development as follows:

Stage 1 – provision of access. Initially, it is necessary to establish a road network to at least provide a basic means of communications between centres of population and between farm and market. At this stage, little attention is paid to geometric standards as it is much more important to consider whether a road link exists at all or, if it does, whether it is 'passable' at all times.

Stage 2 – provision of additional capacity. The next stage is to build capacity into the road network. Geometric standards probably have little to contribute to this except in the areas of road width and gradient. Much more important factors are whether or not a road is paved or whether it has sufficient structural strength to carry the traffic wishing to use it.

Stage 3 – increase of operational efficiency. This final stage is to consider operational efficiency of the traffic and it is at this stage that geometric standards become really important.

Design standards

Developing countries, by their very nature, will not usually be at Stage 3 of this sequence; indeed most will be at the first stage. However, design standards currently in use are generally developed for countries at Stage 3 and they have been developed for roads carrying relatively large volumes of traffic. For convenience, these same standards have traditionally been applied to low-volume roads which lead to uneconomic and technically inappropriate designs.

For roads whose objective is to provide fundamental access (Stage 1), absolute minimum standards can be used to provide an engineered road. The choice of standards will be governed only by such issues as traction requirements, turning circles and any requirements for the road to be 'all weather'.

If the objective of the project is to provide additional capacity for the road (Stage 2), then decisions will need to be taken on whether or not the road should be paved and on what will be the appropriate road width. Some studies have suggested that, for relatively low traffic volumes, road width in excess of five metres cannot be justified in terms of accident reductions or traffic operations. However, a wider cross-section may be appropriate on sections where restrictions on sight distance apply and in sharp bends in order that such curves can be negotiated with ease by the heaviest anticipated vehicle type.

It is only when the objective of the road is to increase the operational efficiency of a route (Stage 3), that comprehensive geometric standards become really important. Standards used by individual developing countries are in most cases based on the American standards issued by American Association of State Highway and Transportation Officials (AASHTO) (ref. 1), with possible modifications to take into consideration the special conditions in the individual country.

Maintenance capabilities

Due to the fact that most developing countries are unable to finance adequate maintenance for their road systems there is a tendency among road designers to adopt high-standard and expensive design solutions assuming that, without appropriate maintenance, a high-standard road will last longer than a poorer one.

However, there is little evidence to suggest that a road designed to a high standard will suffer proportionally less than a road built to an appropriate (lower) standard. Indeed, if maintenance is neglected, the loss sustained on a high-standard road may be greater because the investment is larger.

The proper solution is to ensure that maintenance is adequate to protect the investment made in a road to whatever standard it is designed. Bilateral and multilateral donors must play their part in this by ensuring that any development aid that is provided for road construction includes adequate components for road maintenance.

Ideally, the criterion used to determine geometric design standards should be that of minimizing total cost, i.e. the sum of construction, maintenance and road user costs. For example, the higher the geometric design standards, the flatter must be the gradients on the road, which results in an increase in the earthworks construction costs, but a reduction in vehicles' fuel costs. The aim should be to design the gradients such that the marginal increase in earthworks costs is exactly balanced by savings in vehicle operation costs.

Overseas Road Note 6

A study to develop appropriate geometric design standards for use in developing countries has been undertaken by the Overseas Unit of Transport Research Laboratory (TRL formerly TRRL). The study revealed that most standards currently in use are considerable higher than can be justified from an economic or safety point of view. Geometric design recommendations have been published in Overseas Road Note 6 (ref. 2). Details of the research and criteria used as background to the recommendations are given in ref. (3).

SATCC standards

The member states of the Southern Africa Transport and Communications Commission (SATCC), comprising Angola, Botswana, Lesotho, Swaziland, Mozambique, Zimbabwe, Zambia, Malawi and Tanzania, have agreed on harmonization of standards and specifications for road design, construction and maintenance within the nine member states. Carl Bro International has assisted the nine SATCC countries and *Recommendations on Road Design Standards* to be incorporated in the national standards was issued in 1990 (ref. 4). The main objective of these recommendations was to develop a set of standards for the regional trunk road network which could be accepted by all member countries. The recommendations may therefore be regarded as a compromise between the existing design standards in the region.

Road classification

Geometric standards for road design depend on the functional requirements of the road network, which therefore is divided into various classes of roads. Such classification is also useful for administrative purposes and the rural roads may, as mentioned in Chapter 1, be divided into trunk roads and rural access roads. In the above-mentioned Overseas Road Note 6 rural access roads are classified into three groups.

Access roads

Access roads are the lowest level in the network hierarchy. Vehicular flows will be very light and will be aggregated in the collector road network. Geometric standards may be low and need only be sufficient to provide appropriate access to the rural agricultural, commercial and population centres served. Substantial proportions of the total movements are likely to be by non-motorized traffic.

Collector roads

Collector roads have the function of linking traffic to and from rural areas, either direct to adjacent urban centres, or to the arterial road network. Traffic flows and trip lengths will be of an intermediate level and the need for high geometric standards is therefore less important.

Arterial roads

Arterial roads are the main routes connecting national and international centres. Traffic on them is derived from that generated at the urban centres and from the interurban areas through the collector and access road systems. Trip lengths are likely to be relatively long and levels of traffic flow and speeds relatively high. Geometric standards need to be adequate to enable efficient traffic operation under these conditions, in which vehicle-to-vehicle interactions may be high.

Whilst this hierarchy of access, collector and arterial roads is simplistic, there will in practice be many overlaps of function and clear distinctions will not always be apparent on functional terms alone.

The hierarchy should not be confused with the division of administrative responsibilities which may be based on historic conditions.

7.2 DESIGN CLASSES

Design speed

Most countries design their roads according to a design speed varied depending on functional class of road and type of terrain. There is some evidence that the concept of design speed is not very appropriate as the basis for geometric design as it will often lead to uneconomic designs; cf. ref. (5).

Designs should be justified economically and the optimum geometric standards will vary with both construction and road user costs. Construction costs will be related to terrain type and choice of pavement while road user costs are directly related to the level and composition of traffic.

Design class

It is therefore recommended that the basic parameters of road function, terrain type and traffic flow are defined initially. On the basis of these parameters, a design class is selected, while design speed is used only as an index which links design class to the design parameters of sight distance and curvature to ensure that a driver is presented with a reasonably consistent speed environment.

Table 7.1 shows the design classes and design speeds recommended in Overseas Road Note 6 in relation to road function, volume of traffic and terrain. The table also contains recommended standards for carriageway and shoulder width and maximum gradient which is discussed in later chapters.

The terrain classification as 'level', 'rolling' or 'mountainous' may be defined as average ground slope measured as the number of five-metre contour lines crossed per kilometre on a straight line linking the two ends of the road section as follows:

- level terrain: 0–10 ground contours per kilometre;
- rolling terrain: 11–25 ground contours per kilometre;
- mountainous terrain: >25 ground contours per kilometre.

Table 7.2 shows the design speed according to the recommendations for the SATCC countries.

7.3 SIGHT DISTANCE

The driver's ability to see ahead contributes to safe and efficient operation of the road. Ideally, geometric design should ensure that at all times any object on the pavement surface is visible to the driver within normal eye-sight distance. However,

Table 7.1 Road design standards (TRRL Overseas Road Note 6, ref. 2).

Road Function	*Design Class*	*Traffic Flow* (ADT)*	*Surface Type*	*Width (m)*		*Maximum Gradient (%)*	*Terrain/Design Speed (km/h)*		
				Carriage-way	*Shoulder*		*Mountainous*	*Rolling*	*Level*
Arterial	A	5000–15000	Paved	6.5	2.5	8	85	100	120
	B	1000–5000	Paved	6.5	1.0	8	70	85	100
Collector	C	400–1000	Paved	5.5	1.0	10	60	70	85
	D	100–400	Paved/ Unpaved	5.0	1.0†	10	50	60	70
Access	E	20–100	Paved/ Unpaved	3.0	1.5†	15	40	50	60
	F	<20	Paved/ Unpaved	2.5/3.0	Passing Places	15/20	N/A	N/A	N/A

*The two-way traffic flow is recommended to be not more than one Design Class step in excess of first year ADT.
†For unpaved roads where the carriageway is gravelled, the shoulders would not normally be gravelled; however, for Design Class D roads, consideration should be given to gravelling the shoulders if shoulder damage occurs.

Table 7.2 Design speed (ref. 4).

Type	*Number of lanes*	*Formation width (m)*	*Design speed (km h^{-1})*			*Class*			
			Flat	*Rolling*	*Mount.*	*A*	*B*	*C*	*D*
I	4	36.0	120	100	80	A			
II	4	23.8	110	90	70	A	B		
III	2	14.4	100	90	60	A	B	C	
IV	2	12.4	100	80	60	A	B	C	
V	2	11.1	90	80	50	A[1]	B	C	
VI	2	9.7	90	80	50	A[1]	B[1]	C	D[2]
VII	2	9.0	80	70	50		B[1]	C	D[2]
VIII	2	7.5	70	70	50			C[1]	D
IX	1	7.0	70	60	40			C[1]	D

Notes: A = principal arterial roads.
B = minor arterial roads.
C = collector roads.
D = access roads.
[1] Classification valid in SATCC countries.
[2] If not paved.

this is not usually feasible because of topographical and other constraints, so it is necessary to design roads on the basis of lower, but safe, sight distances.

There are three different sight distances which are of interest in geometric design:

- stopping sight distance;
- meeting sight distance;
- passing sight distance.

Stopping sight distance

The stopping sight distance comprises two elements: d_1 = the distance moved from the instant the object is sighted to the moment the brakes are applied (the perception and brake reaction time, referred to as the total reaction time) and d_2 = the distance traversed while braking (the braking distance).

Reaction time

The total reaction time depends on the physical and mental characteristics of the driver, atmospheric visibility, types and condition of the road and distance to, size, colour and shape of the hazard. When drivers are keenly attentive as in urban conditions with high traffic intensity, the reaction time may be in the range of 0.5–1.0 seconds while driver reaction time is generally around 2–4 seconds for normal driving in rural conditions. Overseas Road Note 6 assumes a total reaction time of 2 sec., while the SATCC Recommendations are using 2.5 sec.

d_1

The distance travelled before the brakes are applied is:

$$d_1 = \frac{10}{36} \times V \times t$$

where:

d_1 = total reaction distance in m;
V = initial vehicle speed in km h^{-1};
t = reaction time in sec.

d_2

The braking distance, d_2, is dependent on vehicle condition and characteristics, the coefficient of friction between tyre and road surface, the gradient of the road and the initial vehicle speed.

$$d_2 = \frac{V^2}{254(f + g/100)}$$

where:

d_2 = braking distance in metres;
V = initial vehicle speed in km h^{-1};
f = coefficient of longitudinal friction;
g = gradient (%; positive if uphill and negative if downhill).

The determination of design values of longitudinal friction, f, is complicated because of the many factors involved. It is, however, known that f-values are decreasing for higher vehicle speed on wet roads. The design values for longitudinal friction used in Overseas Road Note 6 and in the SATCC Recommendations are shown in Table 7.3.

The calculated stopping sight distances ($d_1 + d_2$) are listed in Tables 7.4 and 7.5.

Meeting sight distance

Meeting sight distance is the distance required to enable the drivers of two vehicles travelling in opposite directions to bring their vehicles to a safe stop after becoming visible to each other. Meeting sight distance is normally calculated as twice the minimum stopping sight distance. It is desirable that meeting sight distance is achieved along the entire length of the road for single-carriageway roads and should generally be provided in case the carriageway width is less than 5.0 m, as safe passing on such narrow roads can only be done at reduced speed.

Passing sight distance

Factors affecting passing (overtaking) sight distance are the judgement of overtaking drivers, the speed and size of overtaken vehicles, the acceleration capabilities of

Table 7.3 Coefficient of longitudinal friction, f (ref. 2 and 4).

Design speed ($km\,h^{-1}$)	*TRRL Overseas Road Note 6*	*SATCC Recommendations*
30	0.60	
40	0.55	0.40
50	0.50	
60	0.47	0.38
70	0.43	
80		0.36
85	0.40	
100	0.37	0.34
120	0.35	0.32

Table 7.4 Stopping sight distance, (d_1 + d_2), on level grade (TRRL Overseas Road Note 6, ref. 2).

Design speed ($km\,h^{-1}$)	*Stopping sight distance (m)*
30	25
40	35
50	50
60	65
70	85
85	120
100	160
120	230

Table 7.5 Stopping sight distance, (d_1 + d_2), for different grades (ref. 4).

Design speed ($km\,h^{-1}$)	*Assumed speed for upgrades ($km\,h^{-1}$)*	*Coefficient of friction*	*Minimum stopping sight distance (m)*				
			Upgrades		*Level grade*	*Downgrades*	
			+6%	*+3%*		*−3%*	*−6%*
40	35	0.40	44	44	44	44	47
60	55	0.38	74	76	79	83	86
80	70	0.36	116	122	126	133	140
100	85	0.34	173	178	185	197	210
120	100	0.32	242	250	260	279	301

overtaking vehicles, and the speed of oncoming vehicles. Driver judgement and behaviour are important factors which vary considerably among drivers. For design purposes, the passing sight distance selected should be adequate for the majority of drivers.

Passing sight distances are determined empirically and are usually based on passenger car requirements. Heavy commercial vehicles require longer times than

cars to complete the overtaking manoeuvre, but on the other hand, commercial vehicle drivers have greater visibility ahead because of their higher eye height. Hence they are able to judge sooner and better whether a gap is suitable or not for overtaking, thus partially offsetting any additional overtaking length that might be required.

There are considerable differences in various standards for passing sight distance due to different assumptions about the component distances in which a passing manoeuvre can be divided, different assumed speeds for the manoeuvre and, to some extent, driver behaviour.

The passing sight distances recommended for use by Overseas Road Note 6 and the SATCC Recommendations are shown in Table 7.6.

Table 7.6 Passing sight distances (refs 2 and 4).

Design speed ($km\,h^{-1}$)	*TRRL Overseas Road Note 6*	*SATCC Recommendations*	
		Normal	*Reduced*
40	Not applicable	110	90
50	140		
60	180	230	180
70	240		
80		420	310
85	320		
100	430	700	520
120	590	1040	770

The passing sight distances recommended for use by the SATCC countries are based on a speed difference of $20\,km\,h^{-1}$ between the passing vehicle and the overtaken vehicle and acceleration rates proposed by AASHTO.

The reduced passing sight distance is based on the assumption that the overtaking vehicle under the initial phase has the possibility to regret and fall back in line with the slow moving vehicle.

The overtaking sight distances recommended by Overseas Road Note 6 also assume that the overtaking vehicle may safely abandon the manoeuvre if an approaching vehicle comes into view. Other design criteria are not specified.

Intersection sight distance

Intersection sight distance is the distance along the main road at which an approaching vehicle must be seen in order to permit a vehicle on the distant intersecting road to cross or merge safely with the traffic on the through road.

The intersection sight distance depends on the design speed, the width of road being crossed and the characteristics of the vehicle crossing or merging on to the main road. Generally, the minimum intersection sight distance can be taken as:

$$S = 2 \times V, \quad \text{for passenger cars}$$
$$S = 3 \times V, \quad \text{for trucks}$$

where:

S = sight distance along main road in metres;
V = design speed in $km\,h^{-1}$.

7.4 TRAFFIC VOLUME

Information on traffic volumes, traffic composition and traffic loading are important factors in the determination of the appropriate standard of a road. The traffic has a major impact on the selection of road class, and consequently on all geometric design elements. The traffic information is furthermore necessary for the pavement design, as well as for the effective and efficient planning and allocation of priorities for the construction and maintenance of the road network.

Average Annual Daily Traffic

For low volume roads the design control is the Average Annual Daily Traffic (AADT) in the 'design year'. For routes with large seasonal variations the design control is the Average Daily Traffic (ADT) during the peak months of the 'design year'. The 'design year' is usually selected as year 10 after the year of opening to traffic.

30th Hour Volume

It is not economically sound to design a facility to be congestion-free every hour throughout the year. However, it has been established that each year the traffic volume often reaches that of the 30th heaviest hour, which is the hourly volume exceeded only 29 hours per year. As a rule it is considered sound practice to design roads to carry this volume, called the '30th-Hour-Volume' or 30th HV.

Design Hourly Volume

Design Hourly Volume (DHV) is then expressed as DHV = AADT × K or DHV = ADT × K, where K is estimated from the ratio of the 30th HV to the AADT from a similar site.

The 30th HV expressed as a fraction of AADT can vary as indicated in Table 7.7.

Table 7.7 Variations in 30th HV (ref. 4).

Traffic condition	*K = 30th HV/AADT*
Heavily trafficked road under congested rural conditions	0.08–0.10
Normal rural conditions	0.12–0.18
Road catering for recreational or other traffic of seasonal nature	0.20–0.30

The higher ratios in Table 7.7 refer to roads with a relatively high concentration of traffic during rush hours or large seasonal variations.

Traffic forecast

Estimation of traffic volume in the design year is discussed in Chapter 3.

7.5 SERVICE VOLUME

The ability to accommodate vehicular traffic is a primary consideration in the planning, design and operation of streets and highways. Capacity can be defined as the maximum number of vehicles per unit of time that can pass over a given road section under the prevailing conditions.

There is very little information from developing countries concerning traffic capacity in relation to roadway, traffic and control conditions. Traffic capacity

assessments in developing countries are therefore normally based on the American Highway Capacity Manual (ref. 6), although this has been developed on the basis of experience gained in the USA.

For two-lane asphalt surfaced roads the Highway Capacity Manual introduces three parameters to describe the service quality: average travel speed, percentage time delay and capacity utilization, and the Manual identifies six so-called service levels designated A–F.

Levels of service

At level A, the highest quality of traffic service occurs and motorists are able to drive at their desired speed of 95 km h^{-1} or higher. For ideal conditions (unconstrained geometric, traffic and environmental conditions) a volume of 420 passenger cars per hour, total for both directions, may be achieved.

As the level-of-service decreases, so will the average travel speed. The drivers will experience more delays, platooning becomes intense and at level-of-service E passing is virtually impossible. At the same time the vehicle density will increase as vehicles crowd closer and closer together.

The highest volume attainable under level-of-service E defines the capacity of the road. Under ideal conditions (no restrictive geometric, traffic or environmental conditions), capacity may reach 2800 passenger cars per hour, total for both directions for a two-lane road. However, operating conditions at capacity are unstable and difficult to predict (Figure 7.1). Reference is made to Section 5.7 in Chapter 5.

Level F represents heavily congested flow with traffic demand exceeding capacity. The flow is unstable with unpredictable characteristics.

The six levels of service are presented in the speed–flow diagram shown in Figure 7.1.

Guidelines for selection of levels of service for the design of different classes of rural roads are given in Table 7.8.

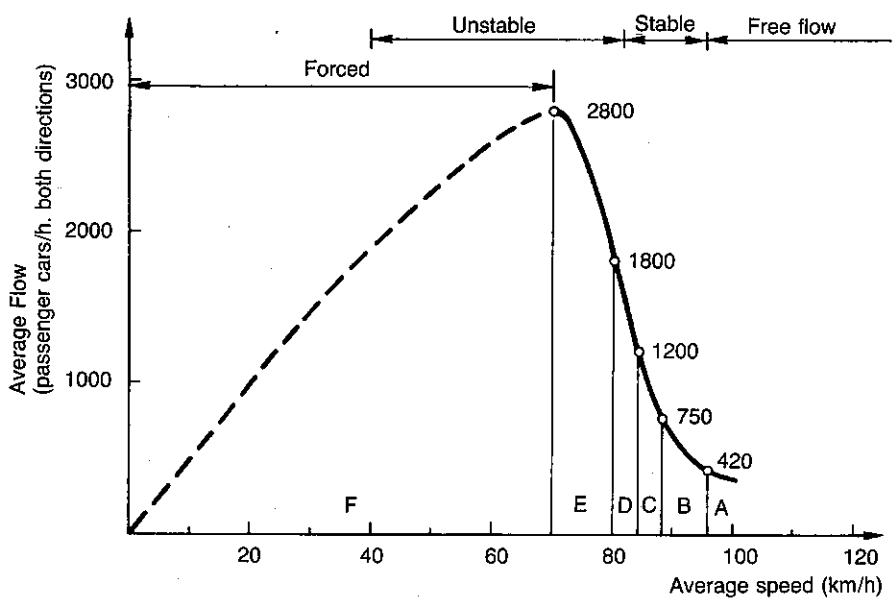


Figure 7.1 Speed–flow diagram.

Table 7.8 Level of service for rural road design (ref. 4).

	Level of service		
Road class	*Flat terrain*	*Rolling terrain*	*Mountainous terrain*
Arterials	B	B	C
Collectors	C	C	D
Local roads	D	D	D

The capacity of two-lane roads varies depending on terrain and the degree of passing restrictions.

To simplify the computations the volume/capacity (V/C) ratios are given in Table 7.9 in terms of the constant 'ideal capacity' of 2800 passenger cars per hour, for level terrain with ideal geometrics and 0% no passing zones, total in both directions.

Table 7.9 Volume/capacity ratio (ref. 4).

Level of service	*Percentage time* delay	*Percentage* no-passing zones*	*Volume/capacity ratio (V/C)*†		
			Flat terrain	Rolling terrain	Mount. terrain
		0	0.15	0.15	0.14
A	30	20	0.12	0.10	0.09
		40	0.09	0.07	0.07
		0	0.27	0.26	0.25
B	≤45	20	0.24	0.23	0.20
		40	0.21	0.19	0.16
		60	0.19	0.17	0.13
		0	0.43	0.42	0.39
		20	0.39	0.39	0.33
C	≤60	40	0.36	0.35	0.28
		60	0.34	0.32	0.23
		80	0.33	0.30	0.20
		0	0.64	0.62	0.58
		20	0.62	0.57	0.50
		40	0.60	0.52	0.45
D	≤75	60	0.59	0.48	0.40
		80	0.58	0.46	0.37

*The percentage of road length where the sight distance is less than 450 m may be used as percentage no-passing zones.

†Ratio of hourly volume to an ideal capacity of 2800 passenger cars per hour in both directions.

Service volume

The service volume, SV, is defined as the maximum volume of traffic that the road is able to serve under the prevailing conditions without falling below the preselected level of service.

The traffic data required for determining the service volume include the two-way hourly volume, the directional distribution of traffic flow and the proportion of trucks and buses in the traffic stream.

SV, for paved two-lane rural roads, can be determined from the general relationship:

$$SV_i = 2800 \times (V/C)_i \times f_W \times f_T \times f_D$$

where:

SV_i = total service volume for level of service i in vehicles per hour;
$(V/C)_i$ = volume/capacity ratio for level of service i;
f_W = lane width factor;
f_T = truck factor;
f_D = directional factor.

The adjustment factor, f_W, for the combined effect of narrow lanes (less than 3.5 m) and restricted shoulder width is given in Table 7.10.

Table 7.10 Lane width factor, f_W (Transportation Research Board, ref. 6).

	Lane width (m)			
Usable shoulder width (m)	*3.5*	*3.35*	*3.00*	*2.75*
≥1.8	1.00	0.93	0.84	0.70
1.2	0.92	0.85	0.77	0.65
0.6	0.81	0.75	0.68	0.57
0	0.70	0.65	0.58	0.49

The truck factor, f_T, is dependent on the level of service, the type of vehicle and the type of terrain and can be determined from the relationship:

$$f_T = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1)}$$

where:

P_T = proportion of trucks in the traffic stream, expressed as a decimal;
P_B = proportion of buses in the traffic stream, expressed as a decimal;
E_T = passenger car equivalent for trucks, obtained from Table 7.11;
E_B = passenger car equivalent for buses, obtained from Table 7.11.

Passenger car equivalents for other vehicle categories and road types are given in Chapter 5.

The V/C values given in Table 7.9 are for a 50/50 directional distribution of traffic. As the directional split moves away from the ideal 50/50 condition the capacity is reduced by the factor f_D as shown in Table 7.12.

For planning purposes the traffic demand is usually expressed in terms of AADT of the design year. The maximum AADT for two-lane rural roads applicable to different levels of service and different types of terrain is given in Table 7.13.

The AADT listed in Table 7.13 is based on an assumed traffic mix of 25%

Table 7.11 Passenger car equivalents (Transportation Research Board, ref. 6).

		Type of terrain		
Vehicle type	*Level of service*	*Flat*	*Rolling*	*Mountainous*
Trucks (E_T)	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
Buses (E_B)	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.5

Table 7.12 Directional factor, f_D (ref. 4).

Directional distribution	50/50	60/40	70/30	80/20
Directional factor, f_D	1.00	0.94	0.89	0.83

Table 7.13 Maximum AADT for two-lane rural roads (ref. 4).

	Maximum AADT		
Level of service	*Flat terrain*	*Rolling terrain*	*Mountainous terrain*
A	1600	700	300
B	3200	1650	700
C	5200	3000	1250
D	8700	4500	1900

trucks, a 60/40 directional split and 20%, 40% and 60% no-passing zones for flat, rolling and mountainous terrain respectively. A ratio of 30th HV to AADT of 0.15 has furthermore been assumed for normal rural conditions as shown in Table 7.7. Recent studies have indicated that passenger-car operation is affected by grades, even where heavy vehicles are not present in the traffic stream. To account for this effect the new edition of *Highway Capacity Manual* (ref. 6) introduces an extra adjustment factor in the general relationship for the total service volume.

Gravel roads

For gravel roads the presence or absence of dust and the roughness of the running surface have a significant influence on the service volume. Although gravel roads are rarely improved for capacity reasons the nominal service volume of a two-lane gravel road can be assumed to be around 10% of that of an asphalt surfaced road with the same alignment and cross-sectional characteristics. Im-

provement of a gravel road to asphalt surfaced (black-top) standard for economic reasons (maintenance and road user cost savings) normally takes place at much lower volumes than the capacity warrants.

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8 Alignment

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8.1 GEOMETRIC DESIGN ELEMENTS

The basic elements of geometric design are: the horizontal alignment, the vertical alignment and the cross-section. The standards to be chosen for these design elements are dependent on the criteria for the design controls as described in the previous chapter, i.e. design speed, sight distance, traffic volume and level of service.

The following elements must be considered when carrying out the geometric design of a road:

1. Horizontal alignment:
   - minimum curve radius (maximum degree of curvature);
   - minimum length of tangent between compound or reverse curves;
   - transition curve parameters;
   - minimum passing sight distance and stopping sight distance on horizontal curves.
2. Vertical alignment:
   - maximum gradient;
   - length of maximum gradient;
   - minimum passing sight distance or stopping sight distance on summit (crest) curves;
   - length of sag curves.
3. Cross-section:
   - width of carriageway;
   - crossfall of carriageway;
   - rate of superelevation;
   - widening of bends;
   - width of shoulder;
   - crossfall of shoulder;
   - width of structures;
   - width of right-of-way;
   - sight distance;
   - cut and fill slopes and ditch cross-section.

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Horizontal and vertical alignment should not be designed independently. They complement each other and proper combination of horizontal and vertical alignment, which increases road utility and safety, encourages uniform speed, and improves appearance, can almost always be obtained without additional costs. It is furthermore important that the choice of standard for the above geometric design elements is balanced so as to avoid the application of minimum values for one or a few of the elements at a particular location when other elements are considerably above the minimum requirements.

8.2 HORIZONTAL ALIGNMENT

The principles in road alignment selection and factors affecting the selection are discussed in Section 8.4, while the horizontal alignment standard to be used is described in this section.

The horizontal alignment should always be designed to the highest standard consistent with the topography and be chosen carefully to provide good drainage and to minimize earthworks. The alignment design should also be aimed at achieving a uniform operating speed. Therefore the standard of alignment selected for a particular section of road should extend throughout the section with no sudden changes from easy to sharp curvature. Where sharper curvature is unavoidable, a sequence of curves of decreasing radius is recommended.

Near-minimum curves should in particular not be used at the following locations:

- on high fills or elevated structures, as the lack of surrounding objects reduces the drivers' perception of the road alignment;
- at or near a vertical curve, especially crest curves, as it would be extremely dangerous, in particular at night time;
- at the end of long tangents or a series of gentle curves; also compound curves, where a sharp curve follows a long flat curve, should be avoided in order not to mislead the driver;
- at or near intersections and approaches to bridges, in particular approaches to single-lane bridges.

The horizontal alignment consists of a series of intersecting tangents and circular curves, with or without transition curves.

Straights

Long straights should be avoided as they are monotonous for drivers and cause headlight dazzle on straight grades. A more pleasing appearance and higher road safety can be obtained by a winding alignment with tangents deflecting some 5–10 degrees alternately to the left and right. Short straights between curves in the same direction should not be used because of the broken back effect. In such cases where a reasonable tangent length is not attainable, the use of long, transitions or compound curvature should be considered. The unfavourable broken back effect may also be improved by the introduction of a sag curve.

The following guidelines may be applied concerning the length of straights:

1. straights should not have lengths greater than (20 × V) metres, where V is the design speed in $km\,h^{-1}$;
2. straights between circular curves turning in the same direction should have lengths greater than (6 × V) metres, where V is the design speed in $km\,h^{-1}$;

3. straights between the end and the beginning of untransitioned reverse circular curves should have lengths greater than two-thirds of the minimum of the total superelevation run-off (see Chapter 9).

Circular curves

As a vehicle traverses a circular curve it is subject to inertial forces which must be balanced by centripetal forces associated with the circular path. For a given radius and speed a set of forces is required to keep the vehicle in its path. The radius can be expressed by the formula:

$$R = \frac{V^2}{127(100e + f_s)}$$

where:

R = radius of curve (metres);
V = speed of vehicle (km h^{-1});
e = crossfall of road (%) (e is negative for adverse crossfall);
f_s = coefficient of side (radial) friction force developed between the tyres and road pavement.

By braking in curves both side and tangential frictional forces are active. The portion of the side friction that can be used with comfort and safety is normally determined as not more than half of the tangential coefficient of friction.

Superelevation

For small radius curves and at higher speeds, the removal of adverse crossfall alone will be insufficient to reduce frictional needs to an acceptable level and crossfall should be increased by the application of superelevation. The maximum value of superelevation is normally set at 7% to take account of the stability of slow, high laden commercial vehicles and the appearance of the road.

However, Overseas Road Note 6 indicates that a superelevation rate of 8% or even 10% may be applied for paved roads under special conditions.

Minimum radius

Table 8.1 shows the minimum horizontal curve radii together with assumed side

Table 8.1 Minimum radii for horizontal curves in metres (refs 1, 2 and 3).

Design speed (km h^{-1})	*TRRL Overseas Road Note 6*			*SATCC Recommendations*		*Bhutan*
	Side friction	*Radius (e = 10%)*	*Radius (e = 7%)*	*Side friction*	*Radius (e = 7%)*	*Radius (e = 7%)*
20						15
30	0.33	15	20			25
40	0.30	30	35	0.20	50	45
50	0.25	60	65		80	75
60	0.23	85	95	0.16	125	120
70	0.20	130	145		180	200
80				0.13	250	280
85	0.18	210	230			
90					335	
100	0.15	320	360	0.11	440	
110					560	
120	0.15	450	515	0.09	710	

friction factors recommended by Overseas Road Note 6 and the SATCC Recommendations for maximum superelevation of 7 and 10%.

The recommended minimum radii of curves below which adverse crossfall should be removed are shown in Table 8.2.

Table 8.2 Minimum curve radii of curve with adverse crossfall in metres (refs 1 and 2).

Design speed ($km\,h^{-1}$)	*TRRL Overseas Road Note 6* $e = -3\%$	*SATCC Recommendations* $e = -2.5\%$
<50	500	1800
60	700	1800
70	1000	2500
80		3000
85	1400	
90		4000
100	2000	5000
110		6000
120	2800	7000

The minimum radius for the design of horizontal curves used in Bhutan, which is also shown in Table 8.1, is based both on the superelevation and side friction considerations as well as the need to ensure stopping sight distance in the curve as illustrated in Figure 8.1. The maximum design values of side friction coefficient used in Bhutan vary from 0.19 at 20–40 $km\,h^{-1}$ down to 0.12 at 80 $km\,h^{-1}$.

Obstructions

Situations frequently exist where an object on the inside of a curve, such as vegetation, building or cut face, obstructs the line of sight. Where it is either not feasible or economically justified to move the object a larger radius curve will be required to ensure that stopping sight distance is available.

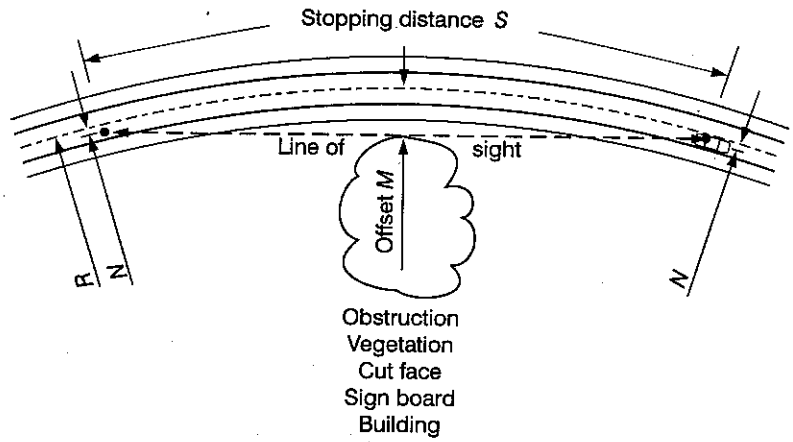


Figure 8.1 Minimum offset to obstruction adjacent to curve.

The required radius of curve is dependent on the distance of the obstruction from the centre-line and the sight distance as shown in Figure 8.1 and can be derived from the relationship:

$$M = \frac{S^2}{8(R - N)} + N$$

where:

M = obstruction to centre-line distance (metres);
R = radius of horizontal curve (metres);
S = stopping sight distance (metres);
N = driver's eye and road object displacement from centre-line in metres (can be assumed to be 1.8 m).

Length of horizontal curve

It is preferable to design the horizontal alignment using a curve radius close to the desirable minimum value for the selected design speed which will minimize the length of the horizontal curve. This type of design provides the maximum length of road where sight distances are not reduced and where overtaking can be carried out. Previous design methods used longer curves to produce 'flowing alignments' and more gentle bends. However, with such designs, sight distances will be restricted on the longer curves and a shorter length of alignment will be available where overtaking is safe.

However, for small changes of direction, it is often desirable to use large radius curves. This improves the appearance of the road by removing rapid changes in edge profile. It also reduces the tendency for drivers to cut the corners of small radius curves.

The use of long curves with a radius near absolute minimum should be avoided where possible, as drivers at speeds other than the design speed will find it difficult to remain in lane. Curve widening reduces such problems (Chapter 9).

Transition curve

The provision of transition curves between tangents and circular curves has the following principal advantages:

1. Transition curves provide a natural easy-to-follow path for drivers, such that the centripetal force increases and decreases gradually as a vehicle enters and leaves a circular curve.
2. The transition between the normal cross slope and the fully superelevated section on the curve can be effected along the length of transition curve in a manner closely fitting the speed–radius relation for the vehicle traversing it.
3. Where the pavement width is to be widened around a sharp circular curve, the widening can conveniently be applied over the transition curve length, in part, on the outside of the pavement without a reverse-edge alignment.
4. The appearance of the highway is enhanced by the application of transition curves.

Transition curves should preferably be used on all superelevated curves for arterial (primary) roads with high design speed. The clothoid which is characterized by having a constantly changing radius is normally considered the most convenient type of transition curve.

For design of transition curves reference is made to the particular literature on this subject.

8.3 VERTICAL ALIGNMENT

The vertical alignment of a road has a strong influence upon the construction cost, the operating cost of vehicles using the road, and in combination with the horizontal alignment also on the number of accidents. The vertical alignment should always be designed to the highest standard consistent with the topography and economy. Preferably it should also be designed to be aesthetically pleasing and due recognition in this regard should be paid to the interrelation of vertical and horizontal curvature (cf. Section 8.1).

The two major aspects of vertical alignment are vertical curvature, which is governed by sight distance and comfort criteria, and gradient, which is related to vehicle performance and level of service.

Gradients

Gradients need to be considered from the standpoint of both length and steepness, and the speed at which heavy vehicles enter the gradient. They should be chosen such that any marginal increase in construction cost is more than offset by the savings in operating costs of the heavy vehicles ascending them over the project analysis period. For access roads with low levels of traffic (AADT of less than 20 or so) it is appropriate to use the maximum gradient that the anticipated type of vehicle can climb safely.

For good traction conditions the maximum traversable gradient is in excess of 20% for four-wheel-drive vehicles, while small commercial vehicles can usually negotiate an 18% gradient and two-wheel-drive trucks can successfully tackle gradients of 15–16% except when heavily laden. These performance considerations have formed the basic limiting criteria for gradients as shown in Table 8.3. These maximum traversable gradients are extremely steep and, if possible, all gradients should be much less severe.

On roads with an earth surface, particular soil types may give rise to slippery conditions during rains and even moderate gradients of around 5% can be very difficult to negotiate. At any tight horizontal curve, in particular hairpin bends, it is desirable to use gradients well below the maximum values given in Table 8.3.

Gradients of 10% or over will usually need to be paved to enable sufficient traction to be achieved, as well as for pavement maintenance reasons.

As traffic flows increase, the reduced economic benefits of more severe gradients,

Table 8.3 Maximum recommended gradient (TRRL Overseas Road Note 6, ref. 1).

Road function	*Design class*	*Maximum gradient (%)*
Arterial	A	8
	B	8
	C	10
Collector	C	10
	D	10
Access	D	10
	E	15
	F	15/20

measured as increased vehicle operating and travel time costs, are more likely to result in economic justification for reducing the severity and/or length of a gradient. On the higher design classes of road (arterials), the lower maximum gradient recommended by Overseas Road Note 6 reflects the economics, as well as the need to avoid the build up of local congestion. However, separate economic assessment of alternatives to long or severe gradients should be undertaken where possible and necessary. An estimation of vehicle operating cost savings may be made from relationships such as those incorporated in the TRL Road Transport Investment Model (RTIM3), or the World Bank's Highway Design and Maintenance Standards Model (HDM.III), cf. Chapter 4.

The cost of construction of a road is generally increasing when the terrain changes from flat to undulating or hilly. The optimization concerning cost of construction and the user benefits, which is related to the road gradient, will therefore justify the use of higher gradients the more undulating and hilly the terrain is. This is reflected in the SATCC Recommendations shown in Table 8.4.

Table 8.4 Maximum gradients depending on type of terrain (ref. 2).

Design speed ($km\,h^{-1}$)	*Maximum gradient (%)*		
	Flat	*Rolling*	*Mountainous*
40	–	–	10
50	–	7	9
60	–	6	8
70	5	5.5	7
80	4	5	6
90	3.5	4.5	–
100	3	4	–
110	3	–	–
120	3	–	–

Sustained grades steeper than 3% have a marked retarding effect on heavy vehicles. Design should aim at grades which will not reduce the speeds of heavy vehicles by more than $25\,km\,h^{-1}$. Critical lengths of constant grade for various reductions in truck operating speed are given in Figure 8.2, assuming that the truck is entering the gradient at $80\,km\,h^{-1}$.

Climbing lane

A climbing lane may be introduced as a more cost-effective alternative where terrain or other physical features preclude shortening or flattening of gradients. Where the length of critical grade is exceeded, consideration should be given to providing a climbing lane.

Benefits from the provision of a climbing lane accrue because faster vehicles are able to overtake more easily, resulting in shorter average journey times and reduced vehicle operating costs. Benefits will increase with increases in gradient, length of gradient, traffic volume, the proportion of trucks, and reductions in overtaking opportunities.

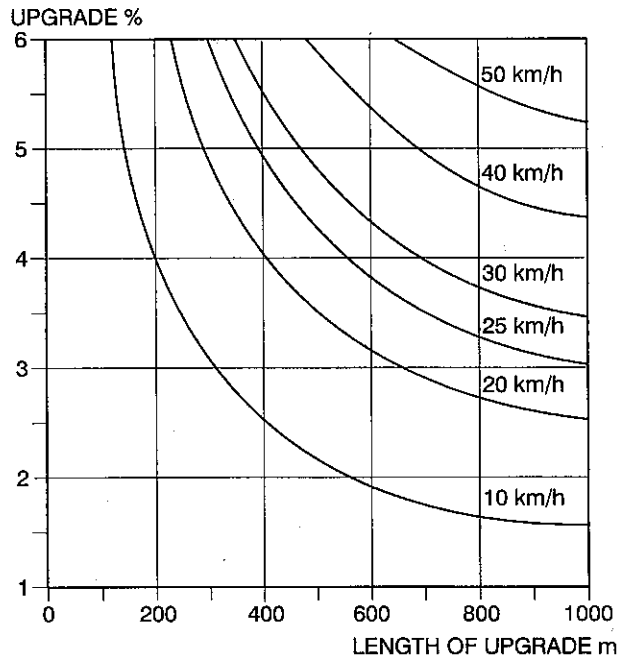


Figure 8.2 Speed reduction effect of upgrade (ref. 4).

Vertical curves

Vertical curves are required to provide smooth transitions between consecutive gradients and the simple parabola is recommended for these. The parabola provides a constant rate of change of curvature, and hence visibility, along its length and has the form:

$$y = \frac{x^2}{2R}$$

where:

y = vertical distance from the tangent to the curve (metres);
x = horizontal distance from the start of the vertical curve (metres);
R = radius of the circular curve that is approximated by the parabola (metres).

x is assumed to be small relative to R.

For practical values of gradient the length of parabolic curve, L_P, can be assumed to be equal to the chord length, L_C, and twice the tangent length, L_T . It may be approximated by the equation:

$$L_P = L_C = 2L_T = \frac{A}{100}R = \frac{i_1 - i_2}{100}R$$

where A is the algebraic difference between the gradients i_1 and i_2 of the intersecting tangents (%).

The rate of change of gradient to successive points on the curve is a constant amount for equal increments of horizontal distance, and equals the algebraic

difference in gradients divided by the length of curve in metres, or A/L_C in per cent per metre. The reciprocal, L_C/A, is the horizontal distance in metres required to effect a 1% change in gradient and is therefore a measure of curvature. The expression $L_C/A = R/100$, termed K, is useful in the determination of the horizontal distance from the start of the vertical curve to the apex of crest curves or to the low point of sag curves. This point where the gradient is zero occurs at a distance from the start of the vertical curve equal to K times the approach gradient.

Crest curves

The most important geometric consideration governing vertical curvature is the sight distance. The driver's view of the road ahead is obscured by the crest vertical curves and, due to the foreshortened view he has of the road, it is not possible for him to assess the severity of the curvature. It is therefore necessary that the driver places a large measure of trust in the designer and assumes that there will not be an isolated vertical curve with a sight distance near minimum in an otherwise high standard design.

Minimum required lengths of crest curves are normally designed to provide stopping sight distance during daylight conditions. Longer lengths would be needed to meet the same visibility requirements at night on unlit roads. Even on a level road, low-beam (meeting) headlight illumination may not show up small objects at the design stopping sight distances. However, higher objects and vehicle tail lights will be illuminated at the required stopping sight distances on crest curves, and it is felt that, since drivers are likely to be more alert at night and/or travelling at reduced speed, these longer lengths of curve are not justified.

If full overtaking sight distance cannot easily be obtained, the design should aim to reduce the length of crest curves to provide the minimum stopping sight distance, thus increasing overtaking opportunities on the gradients on either side of the curve.

A chart of required lengths of crest vertical curves for safe stopping for a 0.2-m high object on the road, and assuming a driver eye height of 1.05 m, is shown in Figure 8.3.

Sag curves

For sag curves the curvature is governed by the visibility at night which is limited by the distance illuminated by the headlamp beams. The minimum curve length, L, for this condition with provision of safe stopping sight distance is given by the relationships:

$$\text{For } S < L: \quad L = \frac{A\,S^2}{200(h + S \times \tan\alpha)}$$

$$\text{For } S > L: \quad L = 2S - \frac{200(h + S \times \tan\alpha)}{A}$$

where:

L = minimum length of vertical sag curve (metres);
S = required stopping sight distance (metres);
h = headlight height (metres);
A = algebraic difference in gradients (%);
α = angle of upward divergence of headlight beam (degrees).

Appropriate values for h and α are 0.6 metres and 1.0 degree respectively.

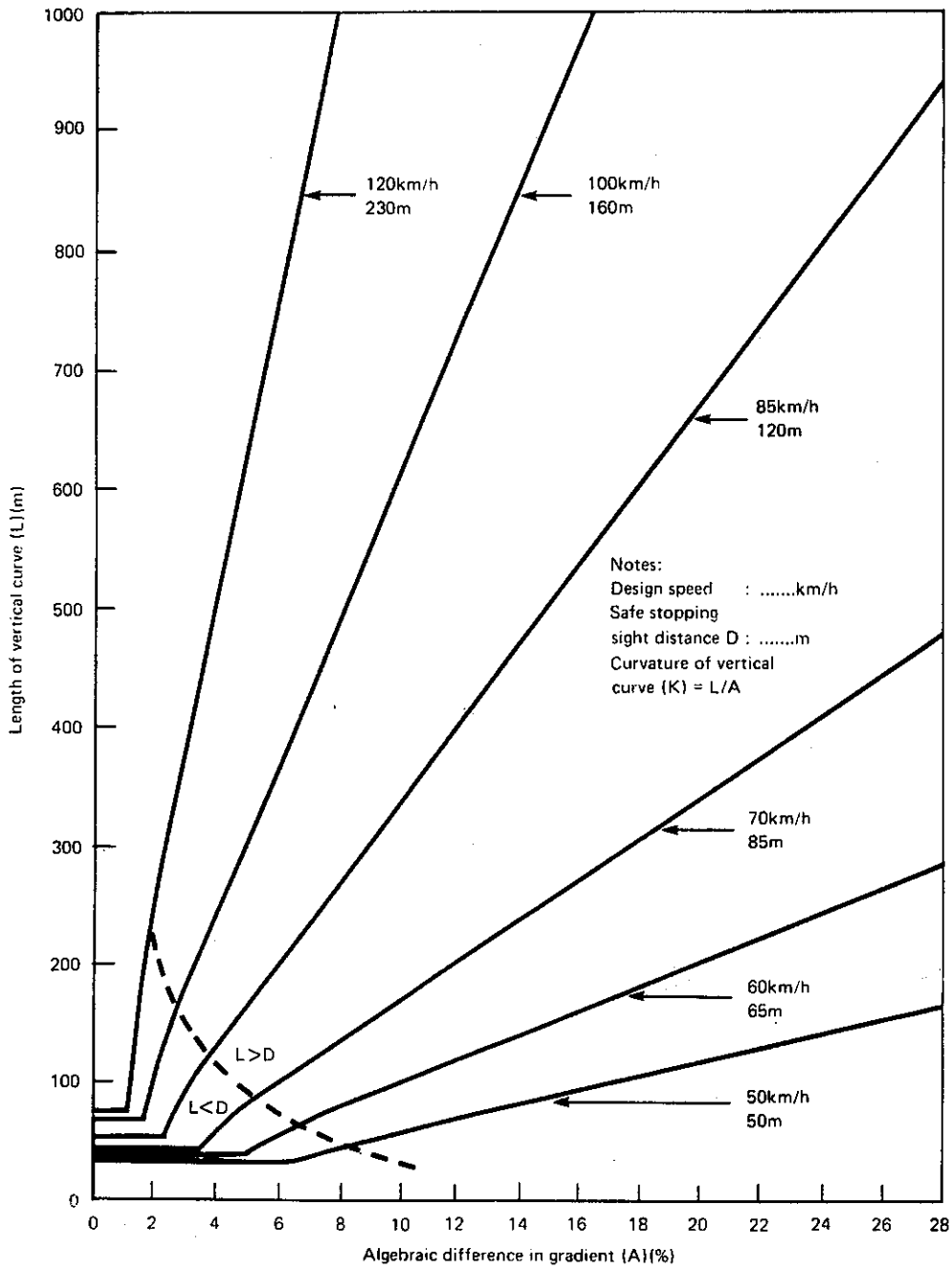


Figure 8.3 Length of crest vertical curves for safe stopping sight distance (TRRL Overseas Road Note 6, ref. 1).

The use of these equations can lead to requirements for unrealistically long vertical curves as, especially at higher speeds, sight distances may be in excess of the effective range of the headlamp beam, particularly when low meeting beams are used. Thus, the only likely situation when the above equations should be considered for use is on the approaches to fords and drifts and other similar locations where flowing or standing water may be present on the road surface.

It is therefore normally more appropriate to design sag curves on the basis of a driver comfort criterion, using a limitation on the vertical acceleration experienced when traversing vertical curves. The minimum lengths of vertical curve based on this criterion are calculated from the following equation:

$$L = \frac{AV^2}{1300C}$$

where:

L = minimum length of vertical sag curve (metres);
A = algebraic difference in gradients (%);
V = design speed ($km\,h^{-1}$);
C = vertical acceleration ($m\,sec^{-2}$).

Values of C recommended by Overseas Road Note 6 are shown in Table 8.5.

Table 8.5 Minimum levels of acceptable vertical acceleration (TRRL Overseas Road Note 6, ref. 1).

Design speed ($km\,h^{-1}$)	120	100	85	70	60	50	40	30
Vertical acceleration in proportion of 9.81 $m\,sec^{-2}$	0.05	0.06	0.07	0.08	0.08	0.09	0.10	0.10

The driver comfort criterion applies to both crest and sag curves. However, on crest curves, this criterion gives much shorter lengths of curve than the stopping distance criterion.

The minimum lengths of sag vertical curves based on the driver comfort criterion are shown in Figure 8.4.

8.4 ALIGNMENT SELECTION

The selection of alignment for a new road is largely a compromise between competing sets of considerations: physical engineering features, safety, aesthetics, environmental and ecological aspects, and the overall costs (construction, maintenance and operating costs). The objective of the designer is to select the alignment which provides an optimum balance between the overall costs and the identified needs of the population to be served by the road. The many factors which influence alignment selection for a road are shown in Figure 8.5.

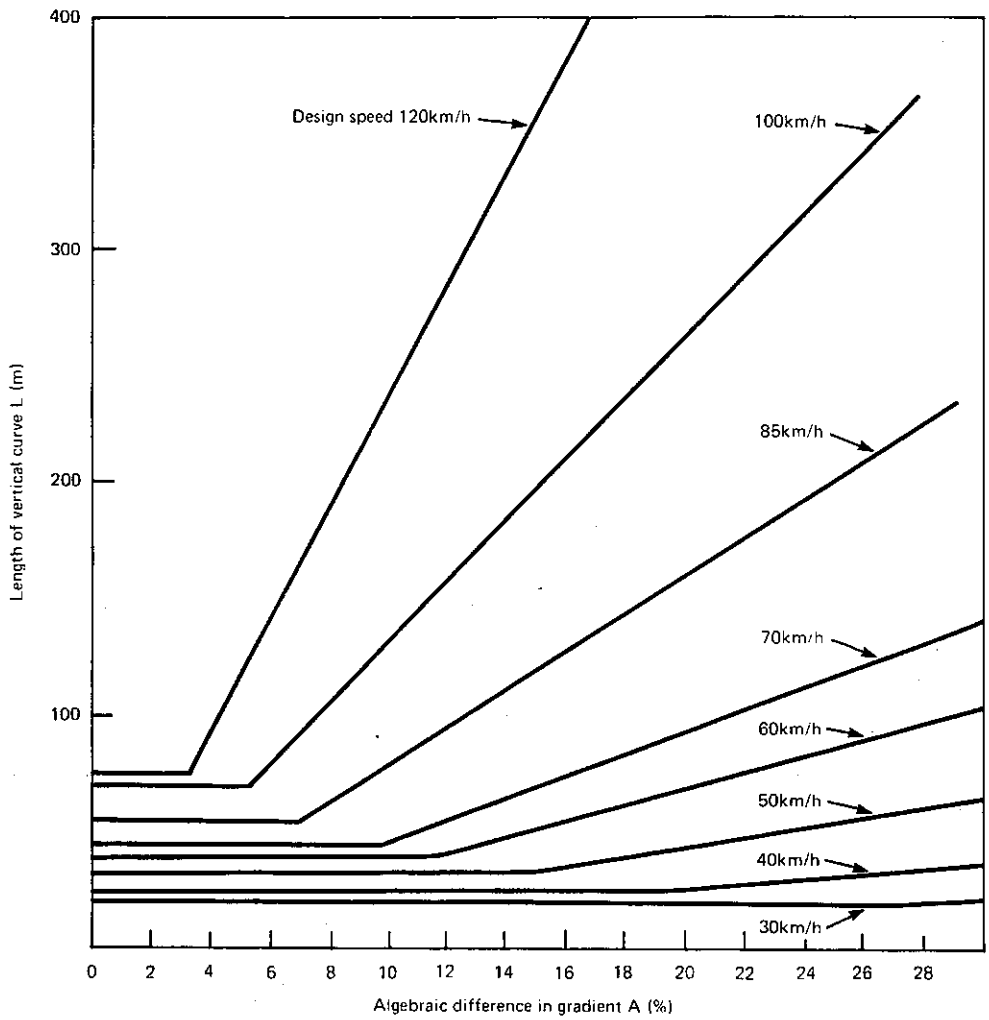


Figure 8.4 Length of sag vertical curve for adequate riding comfort (TRRL Overseas Road Note 6, ref. 1).

Sieve map construction

There are several approaches to selecting the route for a road, but all essentially, either by direct or implicit means, use what is often referred to as the technique of 'sieve map' construction. Assuming that the ends of the route are established and fixed, it is necessary during the reconnaissance study, prior to undertaking any physical surveys, to determine the points of 'sliding fixity'. These are intermediate points of location between the ends which may constrain the route to pass or intersect with a particular feature. Typically, hills, waterways and major transport lines of communications are examined first.

For example, it may be necessary to establish an interchange or junction with a major intersecting road or railway, establish a suitable site for a river crossing, or establish low points (or passes) through a range of hills to be traversed.

Corridor routes

Having established important points of fixity, the first step in building the sieve

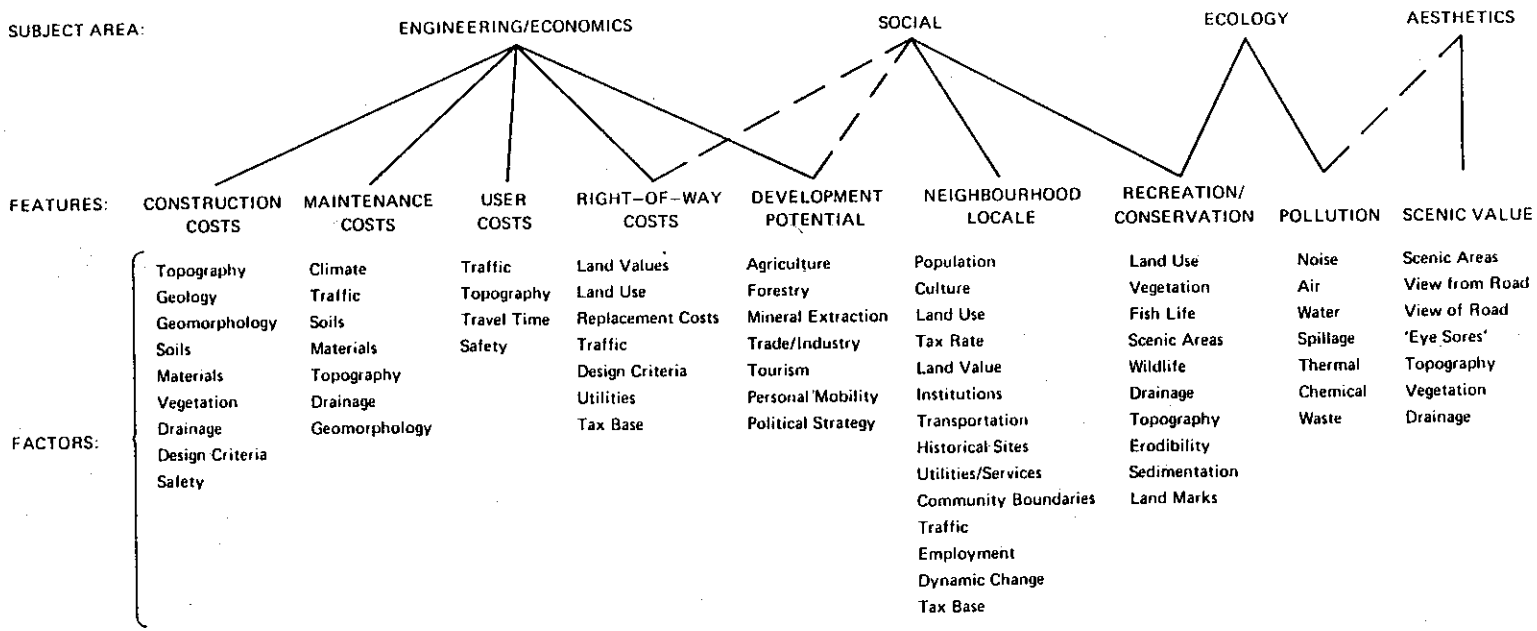


Figure 8.5 Factors affecting road alignment selection (TRRL Supplementary Report 279, ref. 5).

map is to select broad bands of interest within which to select corridor routes for more detailed study. The sieve map consists of a series of traces within each corridor of interest indicating where road construction is undesirable for planning and physical reasons. For instance the planning information trace may identify areas zoned for residential development and/or industrial use; the geological trace may identify areas of peat, marshes, intense gully erosion, areas of potential landslides, mining subsidence, etc., and the drainage trace may identify rivers, streams and canals together with watershed delineation.

Sieve map

Based on the sieve of areas, the route or alternative routes should be selected to avoid as far as possible the more difficult or problematic areas. The choice between different route options depends upon several locational considerations, which normally include but are not limited to:

- minimization of construction costs, including earthworks;
- provision of the shortest route between the fixed end points, thereby minimizing user costs;
- achievement of a suitable level of service fulfilling the needs of the road users;
- provision of a safe and environmentally pleasing route alignment, as far as is possible, from all points of view.

The following aspects will require particular consideration during the alignment study for rural roads:

- topography;
- *in situ* soils and availability of road construction materials;
- drainage conditions and need for major structures for waterway crossings;
- neighbouring projects.

Watersheds

Each of the first three factors can have a significant impact on the construction and maintenance cost of a road project. By selection of a horizontal alignment along primary (or secondary) watersheds the construction and maintenance costs can usually be minimized, but such alignment may be in conflict with the fourth factor. For instance if the road is to serve agricultural areas these are often not located near the watersheds, but in riverine areas with weak subsoils, poor construction materials, swamps and flood-prone areas and with a need for several drainage structures due to waterway crossings, including relief culverts in sidelong terrain. Optimal alignment selection requires a value judgement of these conflicting factors to achieve a satisfactory balance between route length and construction, maintenance and operating costs.

Shortest distance

An alignment close to the shortest distance route will in many situations be the most feasible as for instance pavement construction costs, for routes with identical subsoil conditions, are proportional to the road length. Also operating costs will normally be lower the shorter the route, except if such routes require the introduction of steep grades.

The topography may on the other hand be more gentle along a somewhat longer alignment whereby the earthwork quantities can be reduced and such alignment will often result in the lowest total construction costs. Minimization of earthworks is particularly important if labour-intensive construction methods are to be used.

Co-ordination

During the alignment selection process the road engineer has to carry out co-ordination with a number of different authorities who are responsible for existing

and planned neighbouring projects. Examples of such projects are: other roads, railways, power lines, telecommunication lines, oil pipe-lines, water pipe-lines, agriculture and irrigation schemes, archaeological sites, burial grounds, canals and dam sites, military installations, etc. One of the major constraints in this co-ordination is the lack of information on which projects are actually planned in the area. Furthermore, information on certain projects may be classified, for national security reasons, and therefore be difficult to get access to.

8.5 TOPOGRAPHY AND PHYSICAL FEATURES

Topography, physical features and land use have a pronounced effect on road location and geometrics. In rugged terrain the road location and geometric design elements may be almost completely governed by the topography due to limitations imposed by hills, valleys, steep slopes, rivers and lakes. Information regarding topography and physical features should therefore be obtained in the early stages of planning and design.

Existing maps

In most countries topographical maps, with or without contours, are normally available. The entire country is often covered by maps to the scale 1:50 000 while township maps at 1:2500 or 1:5000 may be available for certain urban areas. However, the maps are often based on 20–40 year old aerial photography and therefore not up to date concerning later infrastructure developments (built-up areas, houses, roads, tracks, telephone lines, power transmission lines, and land use). Consequently such maps are normally only suitable for preliminary investigations and reconnaissance purposes including drainage considerations with catchment area determination. Also geological maps may be available and are useful for the soils and materials investigations.

The search for existing maps begins within the actual country and it may be part of the client's obligations to supply such maps or to assist the consultant in obtaining the permissions required to buy the maps from relevant authorities. Should maps not be available in the country or if they are restricted due to military reasons it will be necessary to try to obtain maps from other sources such as map publishing bureaux like Institut Geographique National (IGN) in Paris or international documentation centres in London and Stuttgart.

In London it is possible to obtain maps to the scale 1:500 000 covering most of the world. These maps, called tactical pilotage charts or 'pilot maps' are produced for aeronautical purposes.

Aerial photos

Aerial photographs, which are generally taken at scales ranging from 1:20 000 to 1:60 000, may be very useful to supplement the existing topographical and geological maps. Their chief advantage is in giving a highly detailed view of the terrain and, when studied with the aid of a stereoscope, the ground surface is seen in full three-dimensional relief.

Satellite imagery

During the past 10 years a new mapping method has been developed, viz. satellite imagery. Landsat satellite imagery is available for the whole world apart from persistently cloudy regions. The images, each covering 185 × 185 km, depict terrain and drainage systems and they are most useful at desk study and project identification stages of investigation. They also show changes in surface features dating back to 1972, by repeated coverage of the same area. Change in major river

flow patterns, retreating coastlines, deforestation or sand dune movements can be observed in this way.

Since early 1986, the French satellite 'SPOT' has also been collecting images of the earth's surface. These are similar to Landsat images but have the advantage of higher resolution, nominally 25 m in colour or 10 m in black and white (i.e. objects larger than 10 × 10 m may be identified). As developments are continuing to improve sensors and observing systems, it is likely that the importance of satellite imagery in providing the highway engineer with relevant information will increase.

Topographic survey

The above-mentioned topographical maps, aerial photographs and satellite images are suitable only for preliminary design of roads. As a basis for detailed design it is necessary to carry out either terrestrial surveys or aerial photography with ground control. Aerial photography is not suitable in areas with dense vegetation throughout the year. With a flight height of 1100 m, corresponding to a photograph scale of approximately 1:7000, maps can be produced with contours having a mean error of 0.33 m and a spot level mean error of 0.22 m. In particular for roads in rolling or hilly/mountainous terrain this accuracy is sufficient for the road design including estimation of earthwork quantities for tender purposes. However, for rehabilitation projects with overlays and some improvements in vertical alignment a terrestrial survey is required.

8.6 INTERSECTIONS

Conflicting vehicle movements at junctions are the largest cause of accidents in many developing countries. A small number of well-designed junctions on a route is preferable to a large number of low-standard junctions. Simple cross-roads have the worst accident record. Staggered cross-roads consisting of two successive T-junctions on opposite sides of the road will reduce the accident rate. The use of roundabouts, traffic lights and channelization may be appropriate to improve vehicle flow and safety. Conflicts can be largely eliminated by the expensive solution of grade separation, but it is not normally necessary to design for free-flow conditions due to traffic capacity requirements. For relatively high traffic volumes it will, especially in urban areas, be more appropriate to improve vehicle flow by traffic light control.

Where traffic lights are required or likely to be installed in the future the intersection spacing should be as regular as possible to permit efficient traffic light phasing on the major road. A guide to the spacing in these conditions is given by:

$$D = 0.139 \cdot C \cdot V$$

where:

D = spacing of intersections (metres);
C = cycle time (sec);
V = vehicle speed ($\mathrm{km\,h^{-1}}$).

Staggered intersections

Where information on the traffic volumes or relative importance of two crossing roads enables the designer to distinguish between the major road and the minor road a staggered intersection should be used wherever possible. In the case of cross intersection angles outside the acceptable range the minor road can be realigned using staggered intersections as shown in Figure 8.6.

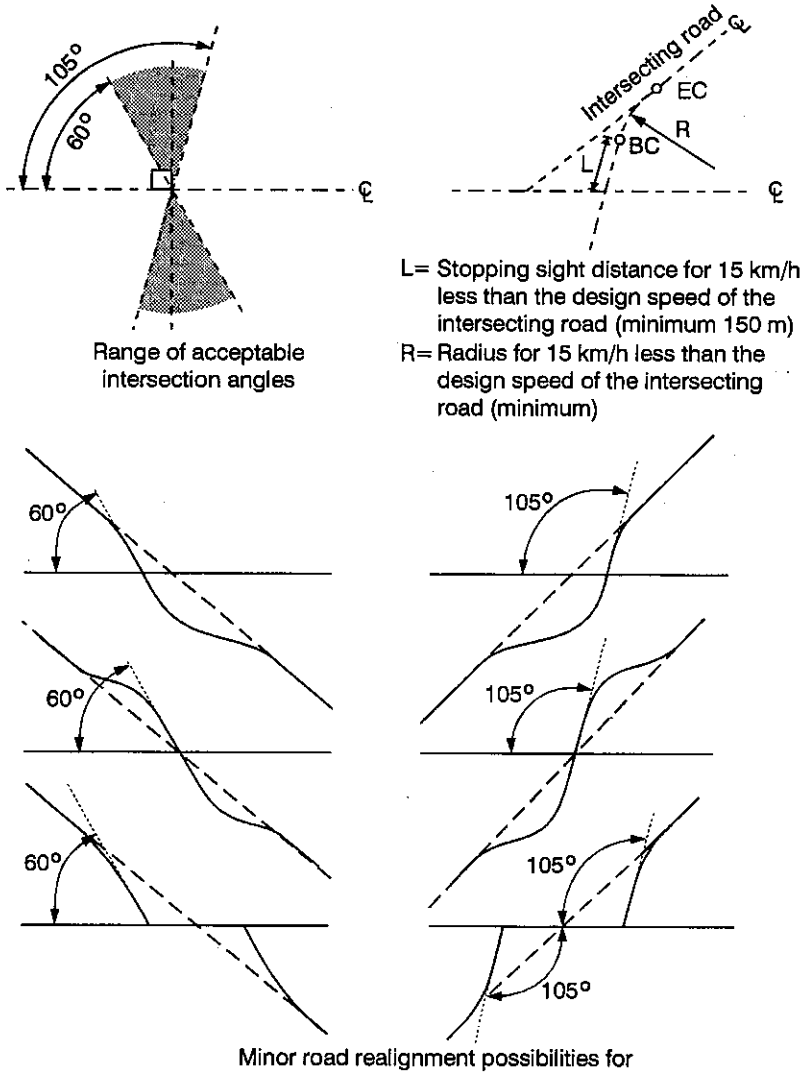


Figure 8.6 Alignment of intersecting roads (ref. 4).

Staggered intersections are referred to as either 'left-right' or 'right-left' according to turning movements of a vehicle on the minor road crossing the major road. Unless the minor road traffic volume is very light, the staggering should be designed so that crossing of the major road traffic is avoided at the exit turn from the major road. Consequently, left-right stagger is not recommended for left-hand driving and right-left is undesirable for right-hand driving.

The minimum spacing of the legs of the staggered intersection should be 100 m. In case of intersections with tapered deceleration/acceleration lanes the minimum spacing should however be 200 m.

Maximum gradient

In order to permit heavy vehicles to operate at reasonable acceleration rates within the vicinity of the intersection, it is desirable that the maximum longitudinal

gradient of any leg of an intersection is 3.0%. This is also the maximum gradient allowed on the intersecting road within 30 m of the point of intersection of the centre-lines of the respective roads.

Intersection layout

Typical intersection layouts for left-hand driving traffic are presented in Figures 8.7, 8.8 and 8.9. The intersections in Figure 8.7 are used for low-volume roads and for property access. These intersections are characterized by no islands or

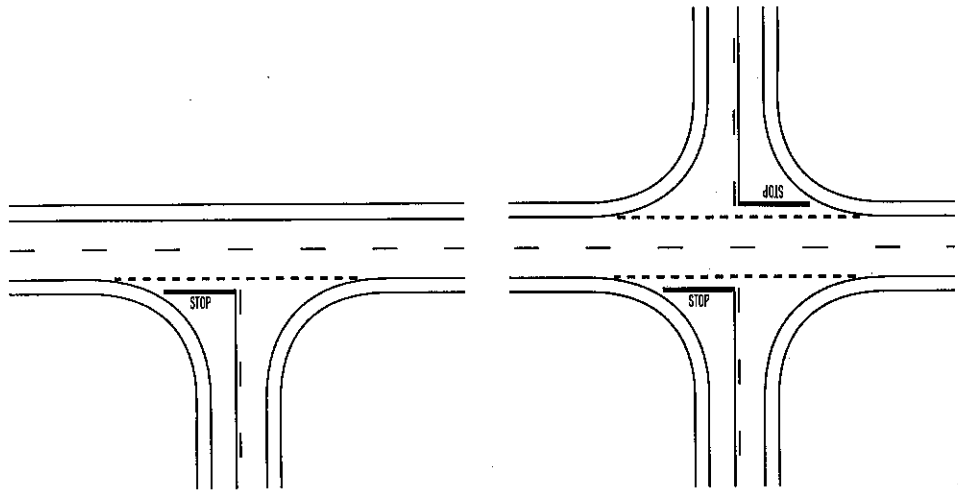


Figure 8.7 Intersection layouts for low-volume roads and for property access (ref. 2).

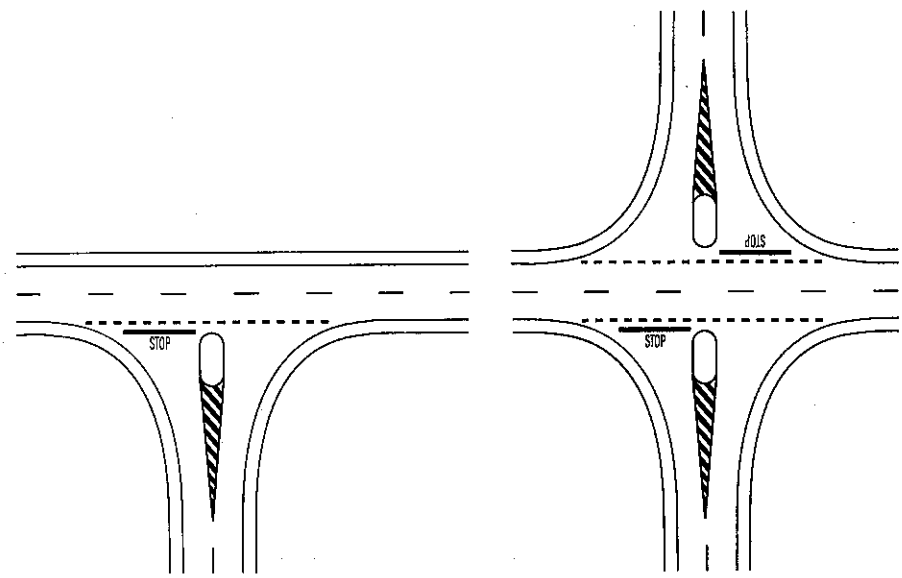


Figure 8.8 Intersection layouts for low-volume roads with light turning traffic from the major to the minor road (ref. 2).

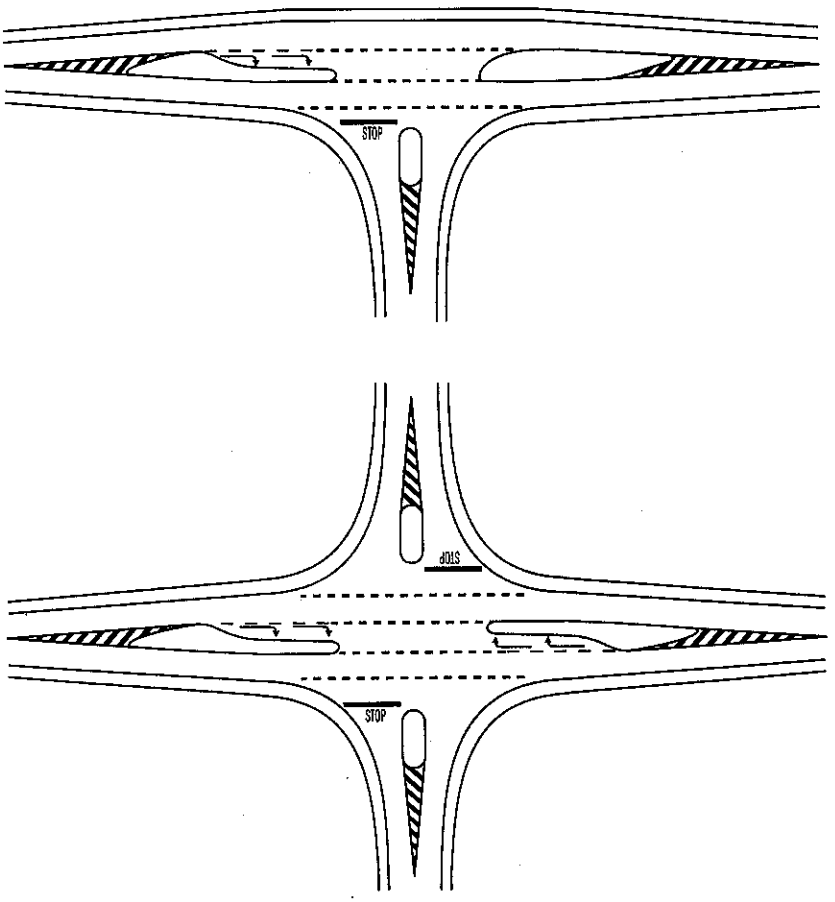


Figure 8.9 Channelized intersection layouts (ref. 2).

widening. The edge of the pavement is constructed as a single circular arc with radius 8–10 m. For private property access the radius is reduced to 6 m.

The intersections in Figure 8.8 are used for low-volume minor roads with light turning traffic from the major to the minor road. They are characterized by the introduction of a divisional island and a widening of the minor road. The edge of the pavement is constructed as a three-point curve consisting of three circular arcs.

The intersections in Figure 8.9 are further provided with a divisional island and channelizing widening of the major road which enables the through traffic to pass waiting right turning vehicles.

Islands

Generally kerbed islands, which are themselves potential hazards, should only be introduced when the hazard which they reduce or eliminate is greater than the hazard which they introduce. Channelization with islands in the middle of the major road (cf. Figure 8.9) should in rural areas be established by painted hatched islands (ghost islands) and not by kerbed islands.

Three-point curve

When the edge of the carriageway is designed with a three-point curve (cf. Figures 8.8 and 8.9), the following ratios between the radii and the centre angles should be used.

$$R1:R2:R3 = 2.5:1:5.5$$
$$A1:A2:A3 = 1:5.5:1$$

The tangent angle A is determined through the following equation (Figure 8.10).

$$A = A1 + A2 + A3$$

The radius R2 of the middle circular arc is chosen according to Table 8.6 depending on the design vehicle.

8.7 LOW-COST ROADS

Major highways are generally designed as 'one of a kind' with heavy engineering involvement in designing construction details warranted by high construction costs and high traffic volumes. The same design-oriented approach to low-volume access roads can easily increase engineering costs to an unacceptable proportion of

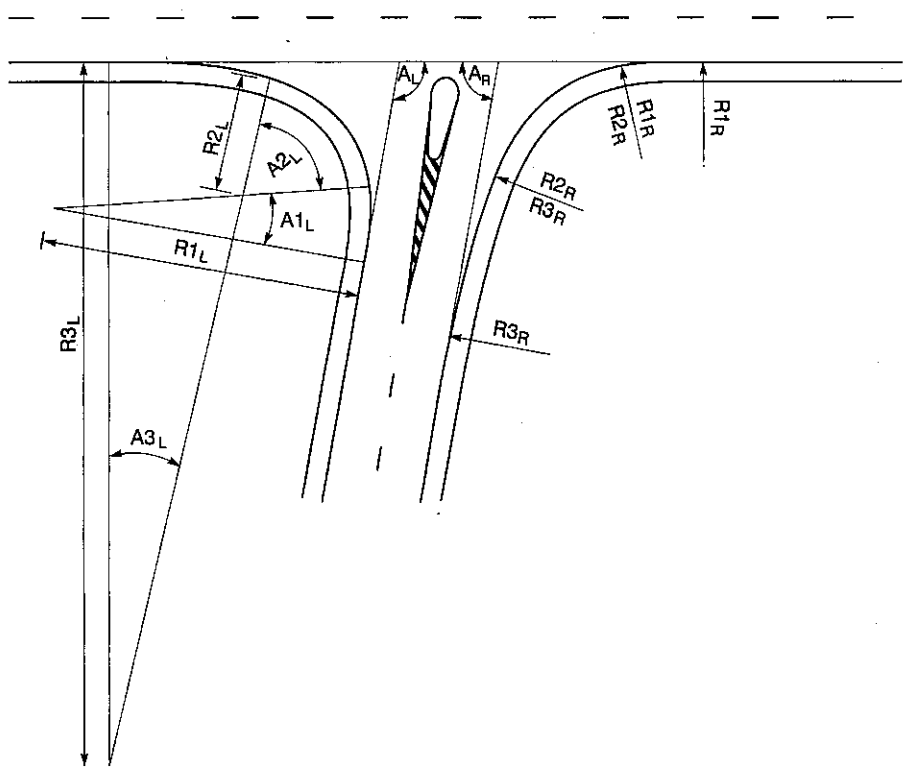


Figure 8.10 Design of the edge of the carriageway with a three-point curve (ref. 2).

Table 8.6 Radius R2 of the middle arc in metres (ref. 2).

	Tangent angle (degrees)			
Design vehicle	*<75*	*75–90*	*90–105*	*>105*
Truck or bus	14	12	12	10
Articulated truck	16	14	10	10

total costs. The field surveys and extent of engineering design should therefore, as far as possible, correspond to the low-cost nature of works required for construction of minor rural access roads.

Definition

Low-cost roads may be defined as those roads that:

1. have a design year AADT of less than 20 vehicles;
2. are constructed without major changes to the natural topography;
3. have no significant quantity of imported manufactured materials used in their construction;
4. are engineered mainly from the *in situ* soils;
5. are constructed on the basis of minimal designs and limited supervision.

Location Plan

Construction of low-cost roads by rehabilitation/upgrading of existing tracks can be done based on a simple Road Location Plan compiled from existing 1:50 000 topographical maps showing roughly the horizontal alignment with kilometre-stationing indicated, including location of existing and potential gravel borrow areas and stone quarries.

Horizontal alignment

Horizontal alignment data are not needed but possible improvements may be indicated in a Schedule of Rehabilitation Works and finally decided during the actual setting out in connection with the construction works.

Schedule

The Schedule of Rehabilitation Works should outline the required earthworks required to form-up the road to the specified standard cross-section, sections requiring gravel wearing course surfacing and required roadside drainage works, cf. Figure 8.11.

Vertical alignment

As the vertical alignment of the rehabilitated road in general will follow closely the ruling grades of the existing ground it will not be necessary to prepare a longitudinal profile. The earthworks involved in forming-up the road prism will therefore in general be quite limited and uniform along the road except on certain sections of the road where cut and fill operations require longitudinal haulage of material. These sections of road – defined as 'cut sections' and 'fill sections' in the Schedule of Rehabilitation Works – occur for instance where improvement of sight distance over crests is required, in hilly and mountainous terrain where relatively large earthworks are needed to establish the typical cross-section or at waterway crossings where fill may be required to suit new drainage structure levels or where the road needs to be raised due to low bearing capacity of the natural soil or in areas liable to flooding.

Earthworks

Only for these cut and fill sections will earthworks be measured in cubic metres during construction, while the forming-up of the road prism to the standard cross-section on the remaining lengths of road will be by the metre of road. This will

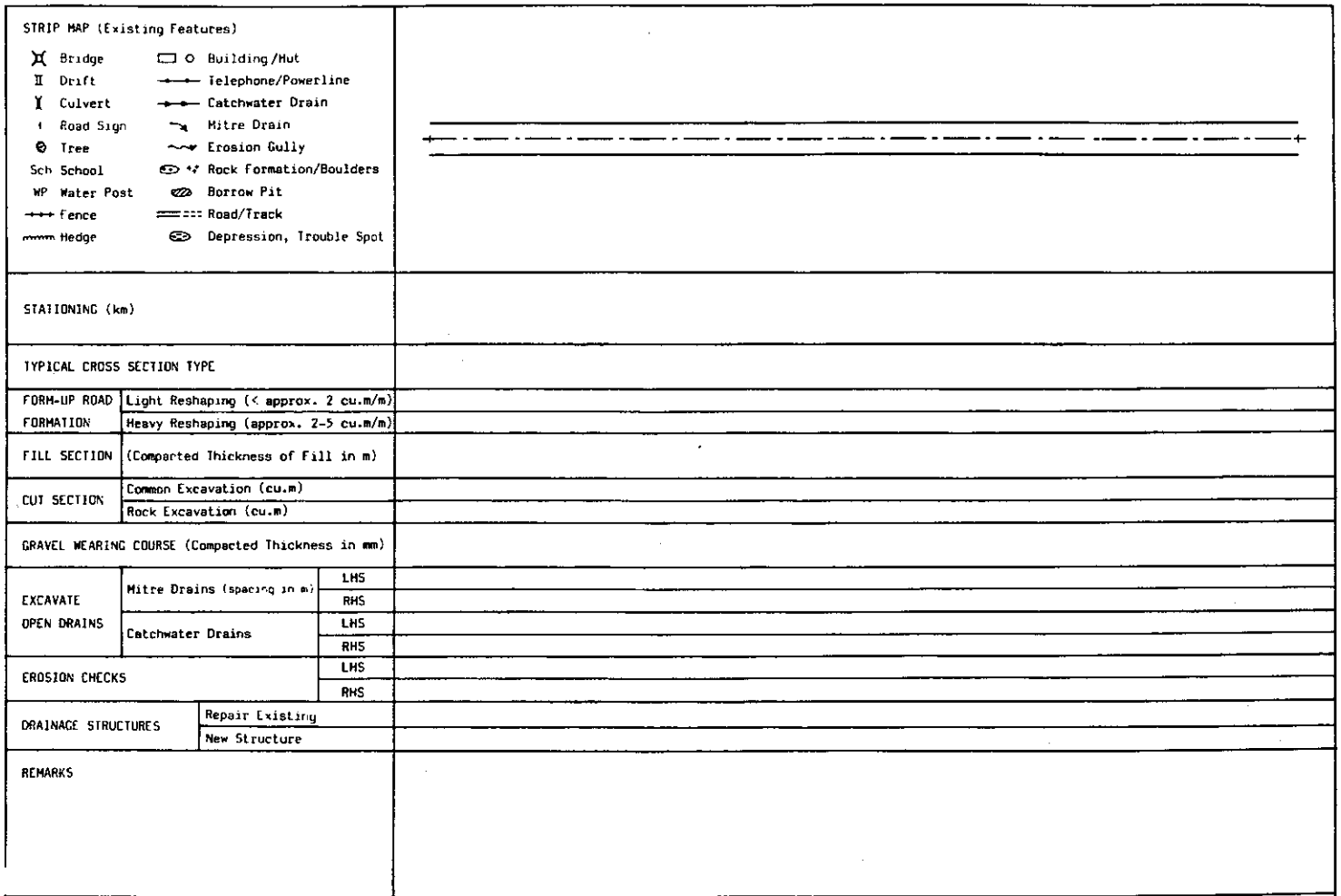


Figure 8.11 Schedule of Rehabilitation Works for rural road improvement (ref. 6).

reduce the need for levelling and other time and manpower demanding surveys to a minimum, thereby meeting the aim of attaining low cost and speed of construction.

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9 Cross-section

Kent Falck-Jensen, Cowiconsult

9.1 ROAD WIDTH

Road width should be minimized so as to reduce the costs of construction and maintenance, whilst being sufficient to carry the traffic loading efficiently and safely.

The following factors need to be taken into account when selecting the width of a road:

1. Classification of the road. A road is normally classified according to its function in the road network. The higher the class of road, the higher the level of service expected and the wider the road will need to be.
2. Traffic. Heavy traffic volumes on a road mean that passing of oncoming vehicles and overtaking of slower vehicles are more frequent and therefore the paths of vehicles will be further from the centre-line of the road and the traffic lanes should be wider.
3. Vehicle dimensions. Normal steering deviations and tracking errors, particularly of heavy vehicles, reduce clearances between passing vehicles. Higher truck percentages require wider traffic lanes.
4. Vehicle speed. As speeds increase, drivers have less control of the lateral position of vehicles, reducing clearances, and so wider traffic lanes are needed.

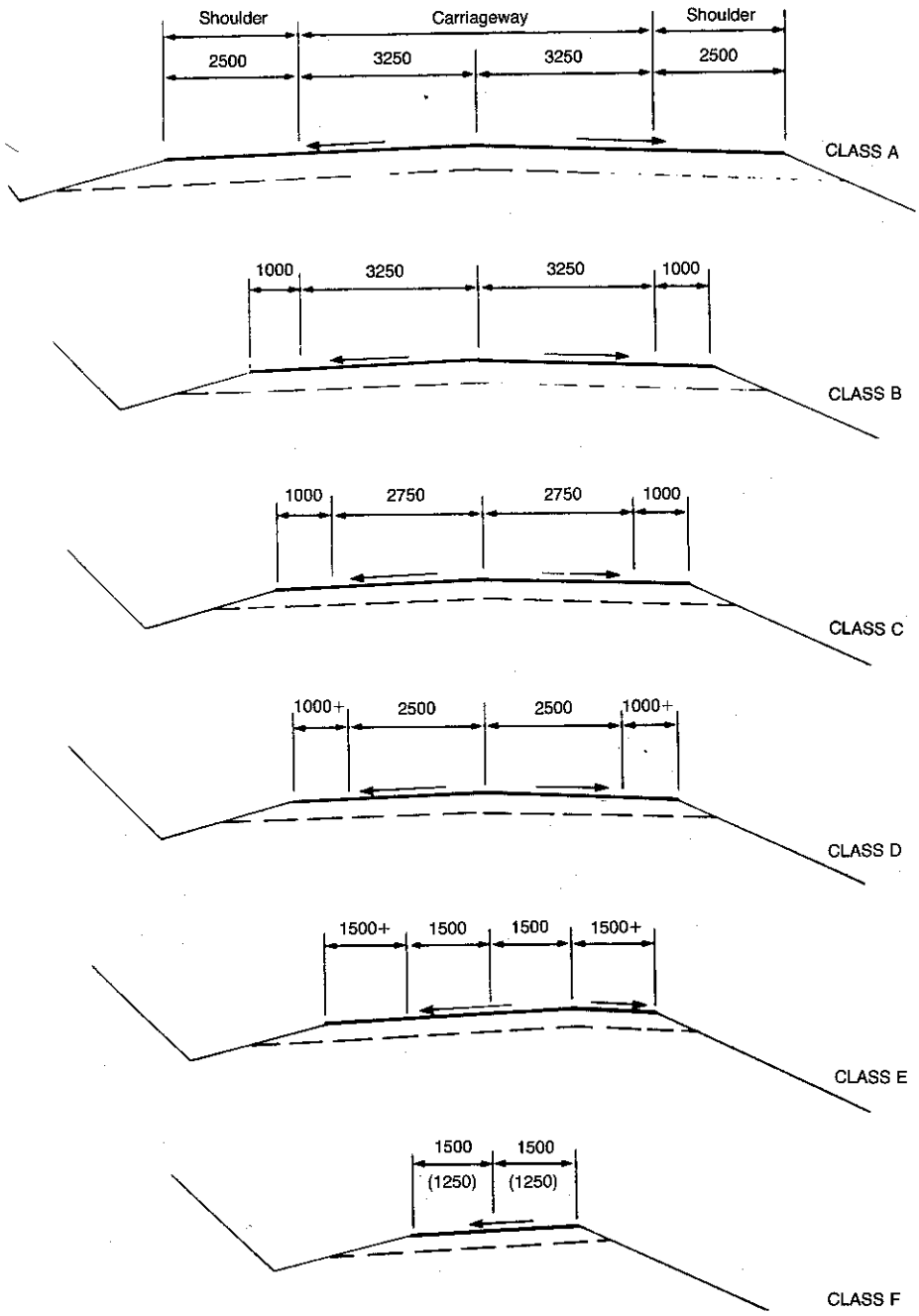
Figure 9.1 shows the typical cross-sections recommended by Overseas Road Note 6 (ref. 1), for the various road design classes A–F. These design classes and related traffic volumes are discussed in Chapter 7.

Access roads

For access roads with low volumes of traffic (<20 AADT) single-lane operation is adequate as there will be only a small probability of vehicles meeting, and the few passing manoeuvres can be undertaken at very reduced speeds using either passing places or shoulders. Provided meeting sight distance is available, these manoeuvres can be performed without hazard, and the overall loss in efficiency brought about by the reduced speeds will be small as only a few such manoeuvres will be involved.

For higher traffic flows (20–100 AADT), single-track roads cause considerable

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1) Dimensions are in mm.
2) Widths should be considered as minimum values and widening is required on curves with tighter radii.
3) Single-lane roads (classes E and F) require meeting sight distance.
4) For a high percentage of heavy vehicles (>40%) it is advisable to increase the running surface width for classes C, D, E and F by 0.50 m.

Figure 9.1 Typical cross-sections (TRRL Overseas Road Note 6, ref. 1).

inconvenience to traffic and it is only to be recommended for short roads or in hilly/mountainous terrain where the cost of construction in side cut and the subsequent haulage cost of materials is high.

Most countries with a rural access road programme have provided roads that are sufficiently wide for two vehicles to pass safely. It should also be noted that, if the new road is to be constructed by machine, the extra construction cost of building a wider road will in level and rolling terrain be quite small.

The Public Works Department in Bhutan, a very mountainous country, recommends single-lane pavements of 3.0–3.5 metres width for traffic volumes up to 200 AADT and with shoulder widths of 0.5–2.0 metres as shown in Table 9.1 (ref. 2).

Table 9.1 Pavement, shoulder and formation widths of rural roads in Bhutan (ref. 2).

		Design speed km h^{-1}	
Road class (design traffic volume)		*up to 60*	*over 60*
Class AA	Pavement	6.0	6.5
Two lanes	Shoulder	1.0	1.5
(>200)	Formation	8.0	9.5
Class A	Pavement	3.5	3.5
One lane	Shoulder	1.5	2.0
(100–200)	Formation	6.5	7.5
Class B	Pavement	3.5	3.5
One lane	Shoulder	1.0	1.5
(50–100)	Formation	5.5	6.5
Class C	Pavement	3.0	3.0
One lane	Shoulder	0.75	1.25
(<50)	Formation	4.5	5.5
Class D	Pavement	3.0	–
One lane	Shoulder	0.5	–
(<25)	Formation	4.0	–

On unpaved roads, the running surface normally extends to the full width of the formation. Where the formation materials have insufficient bearing capacity gravelling should therefore be made in the full formation width.

Where it is anticipated that a paved surface will be provided in the future, consideration should be given to extending the width of the formation at the time of construction to allow for this.

Collector roads

On roads with medium volumes of traffic (100–1000 AADT), the number of passing manoeuvres will increase and pavement widening will become worthwhile operationally and economically. However, in view of the generally high cost of capital for construction in developing countries and the relatively low cost of travel time, reductions in speed when approaching vehicles pass will remain acceptable for such flow levels, and running surface widths of 5.0 and 5.5 metres are recommended by the TRL Overseas Unit. However, in the case of a high

percentage of heavy vehicles (>40%), it is advisable to increase the running surface width to 5.5 or 6.0 metres.

Arterial roads

For arterial roads with higher flows (>1000 AADT), a 6.5-metre wide running surface will allow even heavy vehicles in opposing directions of travel to pass safely without the need to move laterally in their lanes or to slow down.

The standard cross-sections proposed in the SATCC Recommendations (ref. 3) are listed in Table 9.2 and the layout for cross-sections with a paved carriageway are shown in Figure 9.2.

Table 9.2 Standard cross-sections for rural roads (ref. 3).

Road type	*No. of lanes*	*Lane width* W_l *(m)*	*Edge strip* W_e *(m)*	*Carriageway width* W *(m)*	*Shoulder width* W_s *(m)*	*Outer shoulder* W_{os} *(m)*	*Median width* W_m *(m)*	*Road width* W_r *(m)*
I	4	3.5	0.5	2 × 8.0	2.5[1]	1.5	12	36.0
II	4	3.5	0.2	2 × 7.4	2.0[1]	1.0	3[2]	23.8
III	2	3.5	0.2	7.4	2.5[1]	1.0	–	14.4
IV	2	3.5	0.2	7.4	2.0[1]	0.5	–	12.4
V	2	3.35	0.2	7.1	1.5	0.5	–	11.1
VI	2	3.35	–	6.7	1.5	–	–	9.7
VII	2	3.0	–	6.0	1.5	–	–	9.0
VIII	2	2.75	–	5.5	1.0	–	–	7.5
IX	1	3.0	–	3.0	2.0	–	–	7.0

[1] For principal arterial roads emergency lane (2.5 or 2.0 m wide) to be paved.
[2] Median with guardrail.

Culverts and bridges

The cross-section of the road is usually maintained across culverts, but special cross-sections may need to be designed for bridges, taking into account traffic such as pedestrians, cyclists, etc., as well as motor traffic. Reduction in the carriageway width may be accepted, for instance, when an existing narrow bridge has to be retained because it is not economically feasible to replace or widen it. It may also sometimes be economic to construct a superstructure of reduced width initially with provision for it to be widened later when traffic warrants it. In such cases a proper application of traffic signs, rumble strips or speed bumps is required to warn motorists of the discontinuity in the road.

Passing places

For single-lane roads without shoulders passing places must be provided to allow passing and overtaking. The total road width at passing places should be a minimum of 5.0 metres but preferably 5.5 metres, which allows two trucks to pass safely at low speed. The length of individual passing places will vary with local conditions and the sizes of vehicles in common use but, generally, a length of 20 metres including tapers will cater for trucks with a wheel base of 6.5 metres and an overall length of 11.0 metres.

Normally, passing places should be located every 300–500 metres depending on the terrain and geometric conditions. They should be located within sight distance of each other and be constructed at the most economic locations as determined by terrain and ground conditions, such as at transitions from cut to fill, rather than at precise intervals.

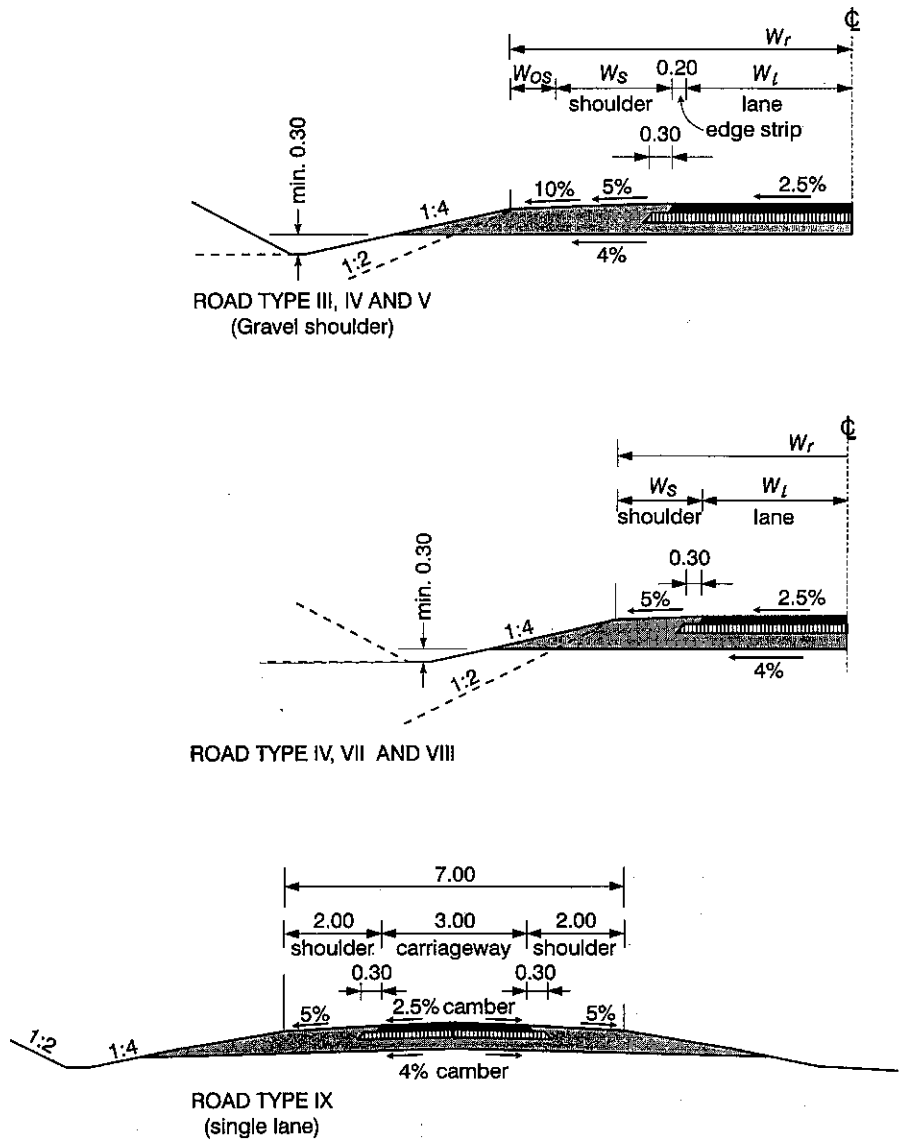


Figure 9.2 Typical cross-section (ref. 3).

Curve widening

The carriageway widths should be increased on low-radius curves to allow for the swept paths of longer vehicles, and the necessary tolerances in lateral location as vehicles follow a curved path. The levels of widening recommended by Overseas Road Note 6 for single-lane and two-lane roads are shown in Tables 9.3 and 9.4 respectively.

The lane widening in curves (widening of each lane) according to the SATCC Recommendations are shown in Table 9.5.

Widening should be applied on the inside of a curve and be gradually introduced over the length of the transition curve or the superelevation development length.

On access roads, which often have substantial horizontal curvature requiring

Table 9.3 Curve widening, single-lane roads with 3.0 m basic width (TRRL Overseas Road Note 6, ref. 1).

Curve radius (m)	20	30	40	60
Increase in width (m)	1.50	1.00	0.75	0.50

Table 9.4 Curve widening, two-lane roads (TRRL Overseas Road Note 6, ref. 1).

Curve radius (m)	<50	50–149	150–299	300–500
Increase in width (m)	1.50	1.00	0.75	0.50

Table 9.5 Curve widening (ref. 3).

	Increase in width (m) for normal lane width (m):			
Curve radius (m)	*2.75*	*3.00*	*3.35*	*3.50*
<50	1.00	1.00	0.65	0.50
50–100	0.75	0.75	0.40	0.25
100–250	0.50	0.50	0.25	
250–750	0.25	0.25		

local widening, it may be practical to increase width over a complete section to offer a more consistent aspect to the driver. This enhancement of the standards should be undertaken where other advantages such as easier construction or maintenance can be identified and where the additional costs are acceptably small.

Shoulders

Shoulders provide for the accommodation of stopped vehicles. Properly designed shoulders also provide an emergency outlet for motorists finding themselves on a collision course and they also serve to provide lateral support for the carriageway. Further, shoulders improve sight distances and induce a sense of 'openness' which improves capacity and encourages uniformity of speed.

In developing countries shoulders are used extensively by non-motorized traffic (pedestrians, bicycles and animals) and a significant proportion of the goods may be transported by such non-motorized means.

Shoulders are recommended on all classes of roads except minor access roads in hilly/mountainous terrain where shoulders would constitute a considerable additional cost. For access roads the combined width of carriageway and shoulders is recommended to be 6.0 metres, which is sufficient for two trucks to pass with 1.0 metre clearance.

A minimum of 1.0 metre of surfaced shoulders is recommended for paved collector and arterial roads to avoid damage and the resulting break up of the edge of the carriageway pavement. For roads with high traffic flow 2.5-metre-wide shoulders should be provided as stopped vehicles blocking any part of the

carriageway would cause a significant hazard. Of the 2.5-metre shoulder, at least one metre should be paved, for instance by a double seal surface treatment.

Paved shoulders shall be clearly segregated from the carriageway either by use of 100-mm-wide carriageway edge line marking or by sealing the shoulders with a different coloured aggregate.

9.2 CROSSFALL AND SUPERELEVATION

Crossfall

Two-lane roads should be provided with a camber consisting of a straight-line crossfall from the centre-line to the carriageway edges, while straight crossfall from edge to edge of the carriageway is used for single-lane roads and for each carriageway of divided roads.

The crossfall should be sufficient to provide adequate surface drainage whilst not being so great as to be hazardous by making steering difficult. The ability of a surface to shed water varies with its smoothness and integrity. On unpaved roads, the minimum acceptable value of crossfall should be related to the need to carry surface water away from the pavement structure effectively, with a maximum value above which erosion of material starts to become a problem.

According to Overseas Road Note 6 the normal crossfall should be 3% on paved roads and 4–6% on unpaved roads.

Due to the action of traffic and weather the crossfall of unpaved roads will gradually be reduced and rutting may develop. To avoid the rutting developing into potholes a crossfall of 5–6% should be reestablished during the routine and periodic maintenance works.

Shoulders having the same surface as the carriageway should have the same cross-slope. Unpaved shoulders on a paved road should be about 2% steeper than the crossfall of the carriageway.

The SATCC Recommendations for crossfall of a paved carriageway and gravel shoulders are given in Figure 9.2.

The normal crossfall on a road will result in vehicles on the outside lane of a horizontal curve needing to develop high levels of frictional force to resist sliding, depending on the speed, curve radius and crossfall. The relationship for this effect is given in Chapter 8, together with recommended minimum radii of curves below which adverse crossfall should be removed.

Superelevation

For small-radius curves and at higher speeds, the removal of adverse crossfall alone will be insufficient to reduce frictional needs to an acceptable level and crossfall should be increased. The preferable maximum value of superelevation is normally set at 7% and with an absolute maximum of 10% to take account of the stability of slow, high laden commercial vehicles and the appearance of the road.

A certain percentage of the centrifugal force is normally compensated by superelevation varying from 30% for high centrifugal force to 80% for low centrifugal force. Superelevation on this basis are shown in Figure 9.3.

On paved roads with unsealed shoulders, the shoulders should drain away from the paved area to avoid loose material being washed across the road.

Where transition curves are used superelevation should be applied over the length of the transition curves. Otherwise it should be introduced such that two-

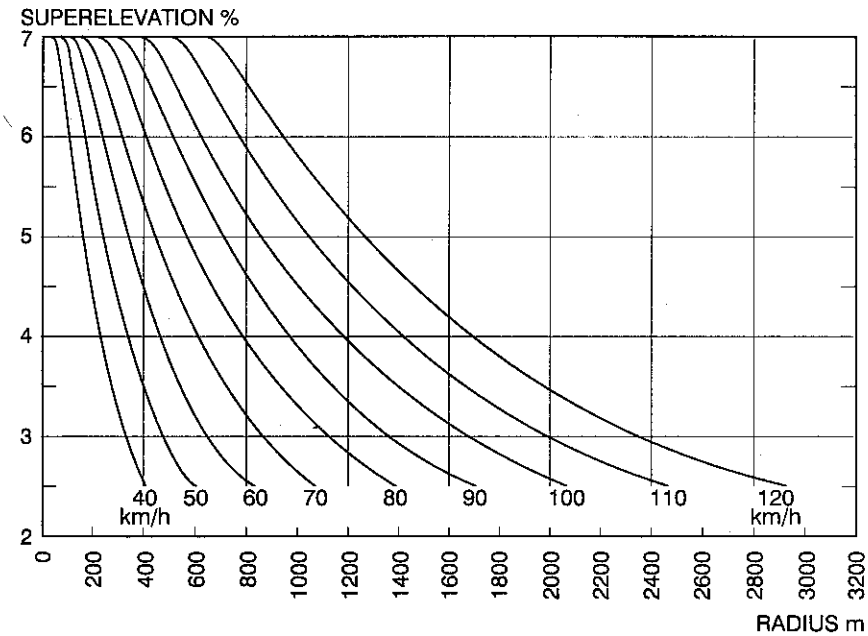


Figure 9.3 Superelevation design curves (ref. 4).

thirds are applied prior to start of the circular curve as shown in Figures 9.4 and 9.5.

The development of superelevation for two-lane roads is normally achieved by raising the outside of the pavement edge relative to the centre-line at a constant relative longitudinal gradient until a straight cross-section is obtained. The whole cross-section is then rotated about the centre-line until the full superelevation is obtained. The relative longitudinal gradient, i.e. the difference between the outside pavement edge and the centre-line, should not exceed a maximum value established from considerations of appearance and comfort criteria.

Superelevation runoff

The transition length from a normal cross-section on a tangent to the fully superelevated cross-section, called the superelevation runoff length, is directly proportional to the total superelevation according to the relationship:

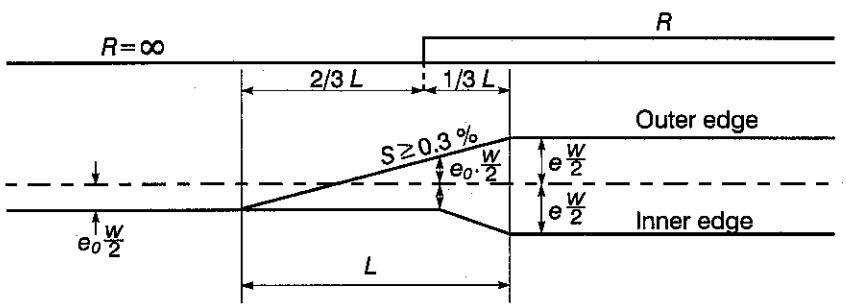


Figure 9.4 Development of superelevation for a circular curve without a transition curve (ref. 3).

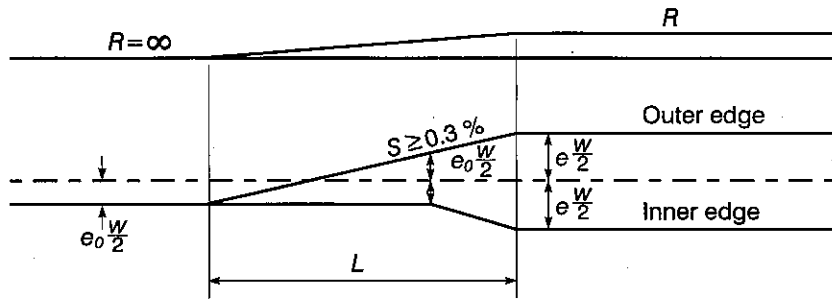


Figure 9.5 Development of superelevation for a circular curve with a transition curve (ref. 3).

$$L = \frac{W}{2S}(e_0 + e)$$

where:

L = superelevation runoff (metres);
W = width of carriageway (metres);
S = constant relative longitudinal gradient (%);
e = superelevation of the curve (%);
e_0 = normal crossfall on the straight (%).

Typical maximum and minimum values for the relative longitudinal gradient, S, are given in Table 9.6.

Table 9.6 Maximum and minimum values for the relative longitudinal gradient (refs 3 and 4).

	Relative longitudinal gradient (%)		
Design speed (km h^{-1})	*Maximum (SATCC)*	*Maximum (Botswana)*	*Minimum*
40 or under	1.50	0.71	0.3
50	1.25	0.67	0.3
60	1.00	0.59	0.3
70	0.75	0.54	0.3
80	0.50	0.50	0.3
90	0.50	0.46	0.3
100	0.50	0.43	0.3

9.3 SIDE SLOPES

The slopes of fills (embankments) and cuts must be adapted to the soil properties, topography and importance of the road. Earth fills of common soil types and usual height may stand safely on slopes of 1 on 1.5 and slopes of cuts through ordinary undisturbed earth with cementing properties remain in place with slopes of about 1 on 1. Rock cuts are usually stable at slopes of 4 on 1 or even steeper depending

on the homogeneity of the rock formation and direction of possible dips and strikes.

Erosion

Using these relatively steep slopes will result in minimization of earthworks, but steep slopes are, on the other hand, more liable to erosion than flatter slopes as plant and grass growth is hampered and surface water velocity will be higher. Thus the savings in original excavation and embankment costs may be more than offset by increased maintenance through the years.

If the construction is to be undertaken by labour-intensive methods it will however be appropriate to minimize earthworks by use of relatively steep slopes as consistent with the angle of repose of the material.

Accident hazard

Steep slopes on fills and inner slopes of side drains create a serious accident hazard. If one wheel of a vehicle goes over the shoulder break point, the driver loses control and may overturn the vehicle. With flat slopes the car can often be directed back on to the road or continue down the slope without overturning.

Suggested cut and fill slopes in earth which generally yield favourable cross-sections are given in Table 9.7.

Table 9.7 Suggested earth slopes for design.

Element	*Access roads*	*Arterial and collector roads*
Inner slope of side drain	1 on 3	1 on 4
Low fills (height $\leqslant$ 3 m)	1 on 2	1 on 4
High fills (height $>$ 3 m)	1 on 1.5	1 on 2
Low cuts (height $\leqslant$ 3 m)	1 on 1.5	1 on 2
High cuts (height $>$ 3 m)	1 on 1	1 on 1.5

Guardrail

A guardrail is recommended to be provided for fills higher than approximately 3 metres. The shoulder is normally widened by 0.5 metres on sections where a guardrail is erected.

Benching

Sometimes in steep terrain the slope of the original ground may exceed 1 on 1.5 (about 34 degrees from the horizontal) and fills will not 'catch' unless unduly extended down the mountain. These long fill slopes may be eliminated by shifting the road into the mountain until the full cross-section is 'benched' into the hillside, but this requires increased excavation with longitudinal hauling of material and increased risk of slides and erosion above the roadway.

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PART 5

Drainage



Multiple culvert in India (Photo by Bent Thagesen)

10 Hydrology

Jan M. Hassing, Cowiconsult

10.1 INTRODUCTION

Impact of water on roads

One of the most important aspects of the design of a road is the provision made for protecting the road from surface water and groundwater. Water on the pavement slows traffic and contributes to accidents from hydroplaning and loss of visibility from splash and spray. If water is allowed to enter the structure of the road, the pavement and subgrade will be weakened, and it will be much more susceptible to damage by the traffic. Water can enter the road as a result of rain penetrating the surface, or as a result of the infiltration of groundwater. When roads fail it is often due to inadequate drainage.

Water can also have a harmful effect on shoulders, slopes, ditches, and other features. Failures can arise spectacularly as for example when cuttings collapse or when embankments and bridges are carried away by flood-water. High water velocities can cause erosion which, when severe, can lead to the road being cut. On the other hand, low velocities at drainage structures can lead to silt being deposited which, in turn, can lead to a blockage. Blockage often results in further erosion or overtopping and possibly wash-out.

Objective of drainage

The objective of surface drainage is to remove stormwater from the roadway so that traffic can move safely and efficiently. In addition cross-drainage structures, including bridges, should be designed to prevent flooding and damage to the roadway and upstream property land.

Design of surface drainage structures involves a choice of maintenance cost and acceptable risks versus construction cost of drainage structures and erosion protection, which often represent a significant proportion of the total road construction costs.

The first step in the drainage design procedure is hydrologic analysis. This analysis shall provide design discharges from all major drainage structures and for rivers and streams adjacent to the road alignment.

Several different types of hydrologic investigations can lead to estimates of the design discharges needed. The type of investigation to be made will depend on the data availability and the possibilities for acquisition of supplementary data.

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Problems

In the context of developing countries, the following problems pertaining to hydrologic design often occur:

- raingauge and discharge station networks are of low density, and the quality and accessibility of data is poor;
- rainfall intensity measurements are few;
- only short data series exist;
- flood (extreme flow) estimation practices are not consolidated;
- land use changes, high-intensity rainfall and erosion causes dynamic changes in the drainage systems, making predictions difficult;
- cost constraints, non-availability of detailed maps and other basic data and difficult logistics limit the possibilities for hydrological surveys.

The following sections are an introduction to the two relevant elements in the hydrological cycle, namely the rainfall and the runoff. They also describe the most common types of investigations and analyses leading to estimates of the flood discharges (extreme flows).

10.2 RAINFALL

Although some regional and global patterns of rainfall can be identified, rainfall is most often unpredictable and highly variable both in time and space. The rainfall volume in a storm is the basic input to many flood estimation methods. The requirements of the method may vary from knowledge of average annual rainfall to knowledge of the detailed variation of rainfall intensity during a rainstorm with a selected recurrence interval.

The extreme events which cause the floods are often storms with a duration of between half an hour to a few hours depending on the catchment size. For larger catchments which have long response times, rainstorms of longer duration will be the critical ones.

Rainfall recording

Rainfall recording takes place using two different types of gauges. One is the daily rainfall gauge, while the other is the autographic gauge. An autographic gauge records both time and accumulated rainfall either on a graph or digitally.

The accuracy of the measurements is influenced by the wind and any obstructions in the vicinity. The wind effect will imply an under-estimation of the real rainfall. The under-estimation can be as high as 10–15% depending on wind speeds and exposure.

Point-area rainfall

The measurements made by the above methods will represent the point rainfall, that is the rainfall at a particular location. Most often, however, one is interested in the total rain falling on a particular area, for instance a selected catchment.

The conversion of point rainfall to area rainfall for a selected standard duration (year, season, month, day or fraction of a day) can be done by various methods. If data from several raingauges are available in the catchment under study, then the following methods can be applied:

- Averaging; a simple average of the amounts of rain at the different gauges for the period of interest.
- Theissen method; a weighted average. On a map neighbouring stations are connected with lines and perpendiculars are raised in the middle of each

connecting line. Each station is supposed to represent an area bounded by the perpendiculars.

- Isohyets (curves representing the same amount of rainfall). When such curves are drawn, areas between the curves can be measured and the total rainfall calculated by multiplying the areas by the mean value of the higher and the lower isohyet.

The isohyetal method is particularly useful in mountain areas, where the isohyets can be drawn to take into account the effects of topography.

Area reduction factors

High-intensity short-duration storms normally only cover areas of a limited size. For areas larger than, say, 10 km^2 this has to be taken into account in rainfall-runoff modelling by applying reduction factors to the point rainfall.

In general, the area reduction factor (the conversion between point rainfall and area rainfall) is dependent on both the area and the duration considered, as well as on the severity of the storm, for instance, expressed as a recurrence interval. However, the dependency on the recurrence interval is often not significant.

An example of an equation for the Area Reduction Factor (ARF) developed for Nairobi in East Africa is:

$$\mathrm{ARF} = 1 - 0.2 \times t^{-1/3} \times A^{1/2}$$

where:

t = rainstorm duration in hours;
A = the area in km^2.

An example of a more general average relationship for a five-year recurrence interval is shown in Figure 10.1.

Recurrence interval

The recurrence interval T is defined as the average number of years between a storm of a given size or larger. The recurrence interval is the inverted probability of occurrence. It should not be inferred that the storm occurs regularly at T-year

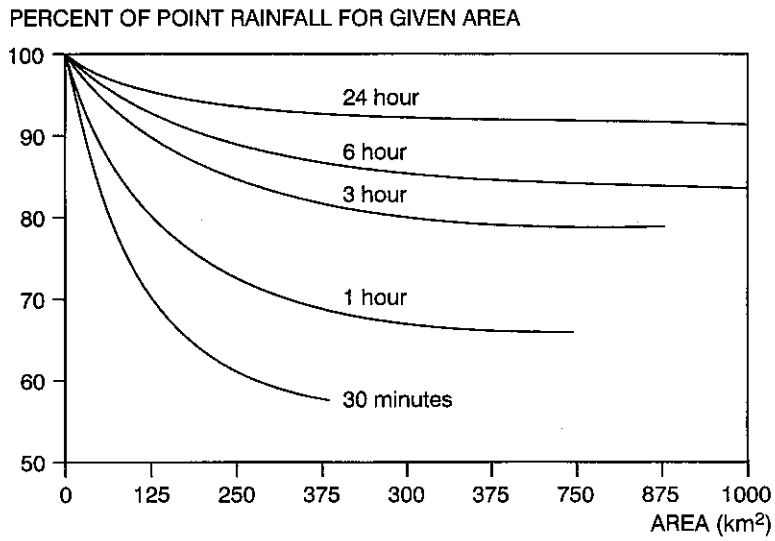


Figure 10.1 Area reduction for a five-year recurrence interval (ref. 1).

intervals, but that it has a probability of occurrence of $1/T$ in any year. It can be shown that the probability of the occurrence of the T-year event within a period of T years is 63%.

Extreme value analysis

When analysing a set of rainfall data the most widely applied tool is an extreme value statistical analysis. Such methods are also applicable to other series of data, for instance discharge data.

Annual maximum series

The analysis is normally performed for an annual series of data. For a selected duration of rainfall, for instance 2 hours, 1 hour or 30 minutes, the largest value is extracted from each calendar year. This series is termed an annual maximum series. The records are listed in order of descending values and a rank is assigned to each of them, the first having rank one, the second rank 2 and so on. Next, the recurrence interval/return period for each value is calculated by the formula:

$$T = \frac{N + 1}{n}$$

where:

T = the recurrence interval;
N = the number of years in the series; and
n = the rank.

The series of annual rainfall extremes can now be plotted against the recurrence interval on probability paper and a curve (straight line) fitted. Extreme values for other recurrence intervals can now be read from the graph or its extrapolation. The best fit is usually obtained by using the Gumbel distribution, but a log normal distribution may in some cases be found applicable. Analytical methods can replace the plotting and the graphs.

Station-year technique

A meaningful statistical analysis requires at least 10 years of reliable records and even then extrapolated values must be considered in a critical sense. If records are shorter than 10 years the station-year technique may be applied. This technique mixes records from several stations assuming that the records are homogeneous and that the events are independent. Records from several stations are then lumped together and considered as a single record and analysed accordingly. The results are then considered valid for all the stations which were the sources of the data. The station-year technique may be justified for rainstorm analyses but cannot be used in an analysis of discharge data. An example of an extreme probability plot is given in Figure 10.2.

Intensity duration relations

It is not very often that adequate rainfall data exists as a basis for the development of a full set of rainfall intensity curves for the location of interest. In such cases it may be possible to use a generalized relation and identify the constants in such a relation. The most common example is:

$$I = \frac{a}{(b + t)^n}$$

where:

I = rainfall intensity in mm h^{-1};
t = the duration;
a, b, and n are constants.

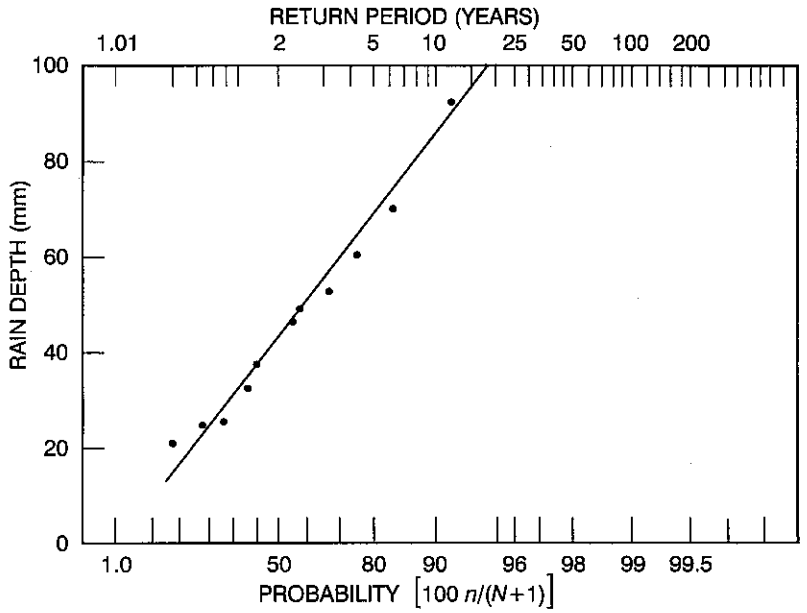


Figure 10.2 Extreme probability plot (ref. 1).

The constants will have to be identified using known values of rainfall intensity-duration and frequency. Examples of intensity-duration curves for different frequencies are shown in Figure 10.3.

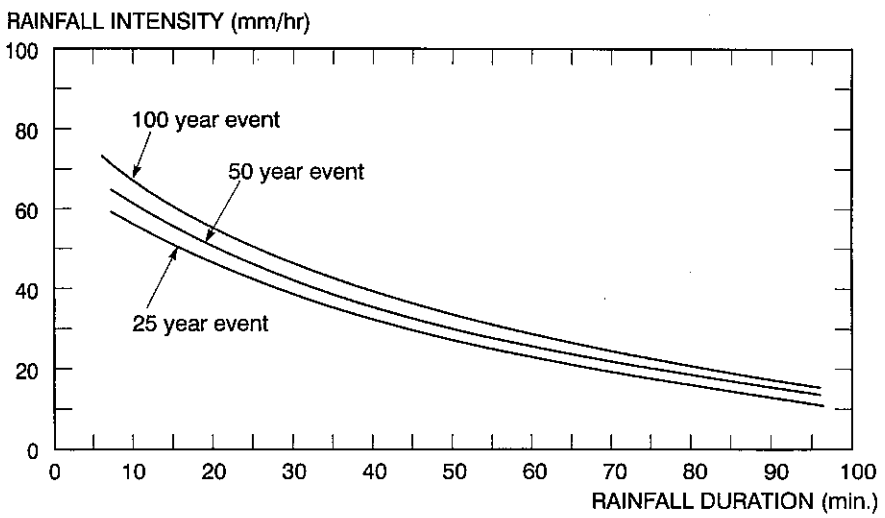


Figure 10.3 Intensity-duration curves.

10.3 FLOOD ESTIMATION – METHODS

10.3.1 Background

Runoff components

The runoff as seen in the rivers and streams is the result of several hydrological processes. In general the runoff comprises the surface runoff, the interflow and the groundwater flow. The surface runoff is the direct result of the excess rainfall, while interflow is the component moving through the soil near the surface. The groundwater component is the contribution from the aquifers through which the stream passes. During a major flood the surface runoff is dominant and usually only the contribution from the excess rainfall has to be taken into account.

Maximum flood discharge

The maximum flood discharge depends on a large number of characteristics of the rainfall, the catchment and the stream. The most significant parameters are the rainfall intensities and their geographical distribution, the catchment area size, the shape and slope of the catchment, the soil infiltration capacity, the vegetation, the slope and roughness of the stream, and the storage capacity of the catchment.

The variation in response between different catchments can be very large. It is therefore necessary to check the similarity of a large number of significant factors before results can be transposed from one catchment to another. Even then the results must be used with caution.

Hydrograph

A plot of the discharge variation over time is termed a hydrograph, and the peak of this hydrograph is the maximum discharge. The shape of the hydrograph varies a lot depending on the climatic conditions and the catchment characteristics. The most peaked hydrographs are found as the flashy responses of mountain catchments, where no vegetation attenuates the surface runoff and where the infiltration capacity is very small. Lower peaks are found in low-relief catchments with extensive vegetation and good infiltration capacity. A hydrograph for a combination of mountain catchments (total area $A = 140\,\text{km}^2$) is shown in Figure 10.4.

Different methods for flood estimation have been developed. They may be divided into three categories:

- analysis of flow data;
- runoff modelling;
- regionalized flood formulae.

10.3.2 Analysis of flow data

Streamflow measurements

The most common form of streamflow measurement comprises a monitoring of the water levels at a suitable cross-section. The relation between water level and discharge at the location is established through a series of discharge measurements (stage-discharge curve). Extrapolations of this curve will give estimates of the peak flood discharge given the flood water level. An extreme value analysis of the annual maximum series will yield estimates of floods for different recurrence intervals.

It is important to distinguish between the momentary peak and the maximum discharge averaged over a day, which is often the discharge quoted in hydrological yearbooks. For smaller catchments there would be a significant difference between these values.

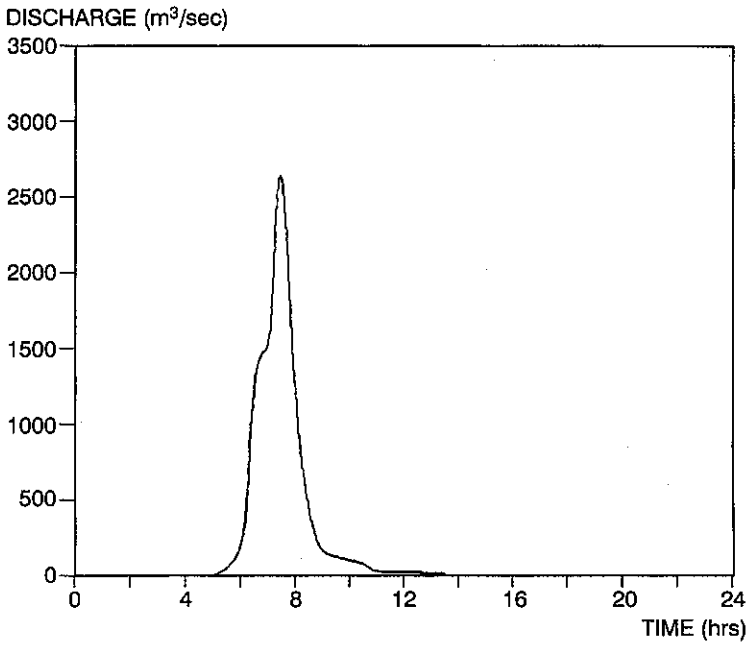


Figure 10.4 A flood hydrograph combining flows from several mountain catchments.

Slope-area method

It is seldom, however, that a discharge gauging station is situated close to the location of interest. Then flood estimates will have to be made from levelling of marks left on vegetation or on the banks, calculation of the cross-sectional area, measurement of the slope of the stream bed and an estimate of the Manning roughness. These observations and estimates combines to yield the flood discharge using the Manning equation:

$$Q = A \times \frac{1}{n} \times R^{2/3} \times S^{1/2}$$

where:

Q = discharge in $m^3\,sec^{-1}$;
A = cross-sectional area in m^2;
n = Manning roughness coefficient;
S = longitudinal slope of the stream bed.

This method for estimating a peak discharge is termed the slope-area method.

Measurements by floats

The surface velocity of the stream can be used in discharge estimates in some cases, where a very rough estimate of the flow suffices. By timing the travel time of a float over a fixed distance a velocity distribution at the surface can be found. With a knowledge of the cross-sectional area, the discharge can be estimated when applying a reduction factor to the surface velocity. The reduction factor can be taken as 0.85.

10.3.3 Runoff modelling

Rational method

The most widely used rainfall runoff relation for ungauged areas is the rational method. It is most suitable for small catchments of sizes up to, say, a few square kilometres.

The basic form of the equation is:

$$Q = \frac{C \times I \times A}{3.6}$$

where:

Q = flood peak at catchment exit ($m^3 sec^{-1}$);
C = rational runoff coefficient (see below);
I = average rainfall intensity over the whole catchment ($mm h^{-1}$) for a duration corresponding to the time of concentration (see below);
A = catchment area (km^2).

Time of concentration

The time of concentration is defined as the time required for the surface runoff from the remotest part of the drainage basin to reach the point considered. The time of concentration can be calculated by the Kirpich formula:

$$T_c = \left(\frac{0.87 \times L^2}{1000 \times S}\right)^{0.385}$$

where:

T_c = time of concentration in hours;
L = length of main stream (km);
S = average slope of main stream ($m m^{-1}$).

Having determined the time of concentration the corresponding rainfall can then be obtained from the intensity-duration curve for the selected recurrence interval (return period).

Runoff coefficient

The runoff coefficient C is an integrated value representing many factors influencing the rainfall runoff relationship, i.e. topography, soil permeability, vegetation cover and land use. The runoff coefficient can be estimated from Table 10.1.

Assumptions

The rational method assumes that:

- the design storm produces a uniform rainfall intensity over the entire catchment;

Table 10.1 Runoff coefficient for the rational method.

Runoff coefficient $C = C_T + C_S + C_V$					
C_T *(topography)*		C_S *(soils)*		C_V *(vegetation)*	
Very flat (<1%)	0.03	Sand and gravel	0.04	Forest	0.04
Undulating (1–10%)	0.08	Sandy clays	0.08	Farmland	0.11
Hilly (10–20%)	0.16	Clay and loam	0.16	Grassland	0.21
Mountainous (>20%)	0.26	Sheet rock	0.26	No vegetation	0.28

- the relationship between rainfall intensity and rate of runoff is constant for a particular catchment;
- the flood peak at the catchment exit occurs at the time when the whole catchment contributes;
- the coefficient C is constant and independent of rainfall intensity.

A more detailed tropical runoff model taking into account all the most significant factors such as area, topography, land use, vegetation, soil type, rainfall intensity and aridity is presented in ref. (1).

10.3.4 Regionalized flood formulae

Flood formulae only including a few parameters have been developed and applied locally and regionally in many cases. Such formulae may be of some use at the particular location but are seldom valid in other environments.

The simplest of these formulae have the general form:

$$Q = C \times A^n$$

where:

Q = flood discharge;
C = a constant;
A = the catchment area;
n = a constant exponent.

A slightly more reliable group of formulae is formed from regression analyses of floods and selected variables in the corresponding catchment. Such formulae will often have the general form:

$$Q = K \times A^m \times B^n \times C^p$$

where:

Q = flood discharge;
K = a constant;
A = catchment area;
B = mean annual precipitation;
C = elevation;
m, n, and p are exponents.

10.4 FLOOD ESTIMATION – FIELD INVESTIGATIONS

In order to estimate rainfall runoff and select the design discharge (design flood), data on rainfall runoff, catchment and river characteristics should be collected to an extent compatible with the project scope. During the data collection it will become evident which flood estimation method will be feasible to apply.

The three methods discussed in the previous section are presented in order of decreasing reliability and also decreasing data requirements. In all cases of flood assessment, estimates must be checked against historic evidence, local experience and practice, research and earlier studies. In spite of considerable efforts, flood

estimation is still quite uncertain and allowances must be made accordingly, when engineering works are being designed.

Analysis of flow data

For the analysis of flow data a reliable streamflow gauging station with more than, say, 10 years of records at or close to the project site is needed. This requirement is only met in a very few cases. However, by similarity, data from neighbouring stations may be transposed and applied in the estimation procedure or for comparison with results of other methods.

Runoff modelling

The rainfall-runoff modelling category may have widely differing data requirements depending on the complexity of the model applied. For the simplest model, only rainfall intensity and a few lumped parameters for the catchment will be necessary. One such model is, for instance, the rational method. The most complex models require a very detailed description of catchment and stream parameters on a grid format as well as the rainfall intensities.

Regionalized flood formulae

The regionalized flood formulae take into account only a few lumped parameters from the catchment while regional empirical constants describe the effects of all other significant factors.

Data information

The information and data relating to the hydrology which ideally has to be collected are:

- topographic maps, scale depending on project scope, but 1:25 000 or 1:50 000 will usually be adequate for identification of the geometric parameters of the catchments;
- air photos or photomosaics from which land use can be studied in the event that land use maps are not available;
- soil and vegetation maps or general descriptions;
- water use in the project area, dams, reservoirs, abstractions for irrigation and other factors which may affect the runoff pattern;
- rainfall data in terms of maximum values of intensities as well as general climatological information;
- discharge gauging station data in terms of annual flood discharges and stage-discharge relation for the purpose of evaluating the accuracy of the flood flow data.

Field information

Further data collection will have to take place in the field. At the sites where bridges or major culverts have to be constructed data on the following should be collected:

- cross-sectional area;
- bed material (in order to estimate the Manning roughness);
- longitudinal slope of stream bed;
- details of historic floods obtained from local residents (in order to estimate peak flood discharges).

River dynamics

With a view to evaluating the possibility of future changes in the runoff regime, registration should also be made of evidence of morphological changes, building up or degradation of river bed levels, erosion patterns, debris size and controls, hydraulic controls, existing drainage structures, evidence of scour, stability of river banks, and land use changes.

10.5 DESIGN DISCHARGE

Selecting recurrence interval

The design discharge for a drainage facility depends on the selected flood frequency or recurrence interval. By selecting a large recurrence interval and a corresponding large design flood, the probability of having such a flood occurring and the risk of damage is reduced, but costs of structures to accommodate such large floods are increased. Conversely, the selection of a small design flood reduces the initial cost of structures and increases the risk of damage from larger floods.

When a recurrence interval is selected for a particular location, the designer is implying that the estimated effects of a larger flood on life, property, traffic and the environment do not justify constructing a larger structure at the time. Risk of serious damage begins when floods exceeding the design flood occur. Damage from lesser or more frequent floods should be minimal and acceptable.

Risk

Risk has been defined by some in terms of the recurrence interval of a flood of stated magnitude being exceeded or the probability of a flood of stated magnitude being exceeded in any one year.

Selecting a design flood frequency by risk as defined above is not the complete answer. The 50-year flood can be exceeded at one location with minimal damage, but at another location, occurrence of the same frequency flood might approach a disaster. Risk and the selection of the design flood recurrence interval involves an evaluation of possible loss of life, property damage, the interruption of traffic, and the economic consequences. Further, aspects like balancing of low construction/high maintenance costs against high construction/low maintenance costs are involved.

Often, design manuals will contain general statements on recommended design flood recurrence intervals. An example for major drainage structures is given in Table 10.2.

Table 10.2 Recommended design flood recurrence intervals for major drainage structures.

Road class	*Recurrence interval (years)*
Expressways	100
Arterial roads	50
Collector roads	50
Access roads	25

It is part of sound practice to supplement design calculations with sensitivity analyses, showing consequences in terms of, for instance, damage for different recurrence intervals.

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11 Hydraulic design

Alan R. Jacobsen, Coviconsult

11.1 INTRODUCTION

Functions of drainage

The drainage system has four main functions:

1. to convey stormwater from the surface of the carriageway to outfalls;
2. to control the level of the water table in the subgrade beneath the carriageway;
3. to intercept groundwater and surface water flowing towards the road;
4. to convey water across the alignment of the road in a controlled fashion.

The first three functions are performed by longitudinal drainage components, in particular side drains, while the fourth function requires cross-drainage structures such as culverts, fords, drifts and bridges.

11.2 LONGITUDINAL DRAINAGE

Crossfall

The road surface must be constructed with a sufficient camber or crossfall to shed rainwater quickly, and the formation of the road must be raised above the level of the local groundwater table.

Wider pavements increase the catchment area, thus increasing the quantity of stormwater that has to be removed. Flatter gradients, both transverse and longitudinal, increase water depth on the surface.

Longitudinal gradients

It is more important to maintain a minimum longitudinal gradient on curbed pavements than on uncurbed pavements in order to avoid undue spread of stormwater on the pavement. However, vegetation along the pavement edge may impede the runoff of water from uncurbed pavement if the gradient is flat. Where the longitudinal gradient of the roadway has to be near zero, the depth of side drains may have to be varied to obtain sufficient gradient of the ditch.

The longitudinal gradients should therefore preferably not be less than 0.3% for curbed pavements, and not less than 0.2% in very flat terrain. Minimum grades can be maintained in very flat terrain by use of a rolling profile or by warping the cross-slope to achieve a rolling gutter profile.

To provide adequate drainage for curbed pavement in sag vertical curves, a

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minimum longitudinal gradient of 0.3% should be maintained within 15 m of the level point in the curve. Zero gradients and sag vertical curves should be avoided on bridges.

Subsurface drainage

Ideally, the base and subbase should extend below the shoulder to the side ditches with a sufficient crossfall of the subbase. When ditches are lined with concrete, etc., drainage outlet pipes/weep holes through the lining must be provided.

Cross-drains

If it is too costly to extend the base and subbase material below the shoulder, cross-drains at 3–5 m intervals should be cut through the shoulder to a depth of 50 mm below subbase level. The cross-drains should be backfilled with base material, or more permeable material, and should be given a fall of 1 in 10 to the side ditch. Alternatively, a continuous drainage layer 75–100 mm thick of pervious material can be laid under the shoulder at the level of the underside of the subbase. Perforated drainpipes can also be used to drain the road pavement.

Roadside drainage

Open roadside drainage channels may be classified according to their function as ditches, gutters, turnouts, chutes, and intercepting ditches.

Ditches

Ditches are channels provided to remove the runoff from the road pavement, shoulders, and cut and fill slopes. The depth of the ditch should be sufficient to remove the water without risk of saturating the pavement subgrade. Ditches may be lined in order to control erosion. Unlined ditches should preferable have side slopes not steeper than 4 horizontal to 1 vertical.

Gutters

Gutters are channels at the edges of the pavement or the shoulder formed by a curb or by a shallow depression. Gutters are paved with concrete, brick, stone blocks, or some other structural material. Spacing between outlets on curbed road sections depends on runoff, longitudinal gradient and permissible water depth along the curb.

Turnouts

Turnouts or mitre drains (Figure 11.1) are short, open, skew ditches used to remove water from the roadside ditches or gutters. Use of turnouts reduces the necessary size of the side ditches and minimizes the velocity of water and thereby the risk of erosion. Turnouts must be provided at intervals depending on runoff, permissible velocity of water and slope of the terrain. To prevent the flow through a turnout from generating soil erosion at its outlet, the discharge end of the turnout should be fanning out.

Chutes

Chutes are open, lined channels or closed pipes used to convey water from gutters and side ditches down fill slopes and from intercepting ditches down cut slopes. On long slopes, closed (pipe) chutes are generally preferable to open chutes. The inlet of chutes must be designed to prevent water bypassing the chute and eroding the slope. The outlet must likewise be designed to prevent erosion at the outlet (Figure 11.2). The chute interval will depend on the capacity of gutters or ditches.

Intercepting ditches

Intercepting (or cut-off) ditches are located on the natural ground near the top edge of a cut slope or along the edge of the right-of-way, to intercept the runoff from a hillside before it reaches the road. Intercepting the surface flow reduces erosion of cut slopes and roadside ditches, lessens silt deposition and infiltration in the road-bed area, and decreases the likelihood of flooding the road in severe storms.

Intercepting surface water is particularly important in arid and semi-arid regions because of generally low water infiltration capacity and high tendency to erosion of

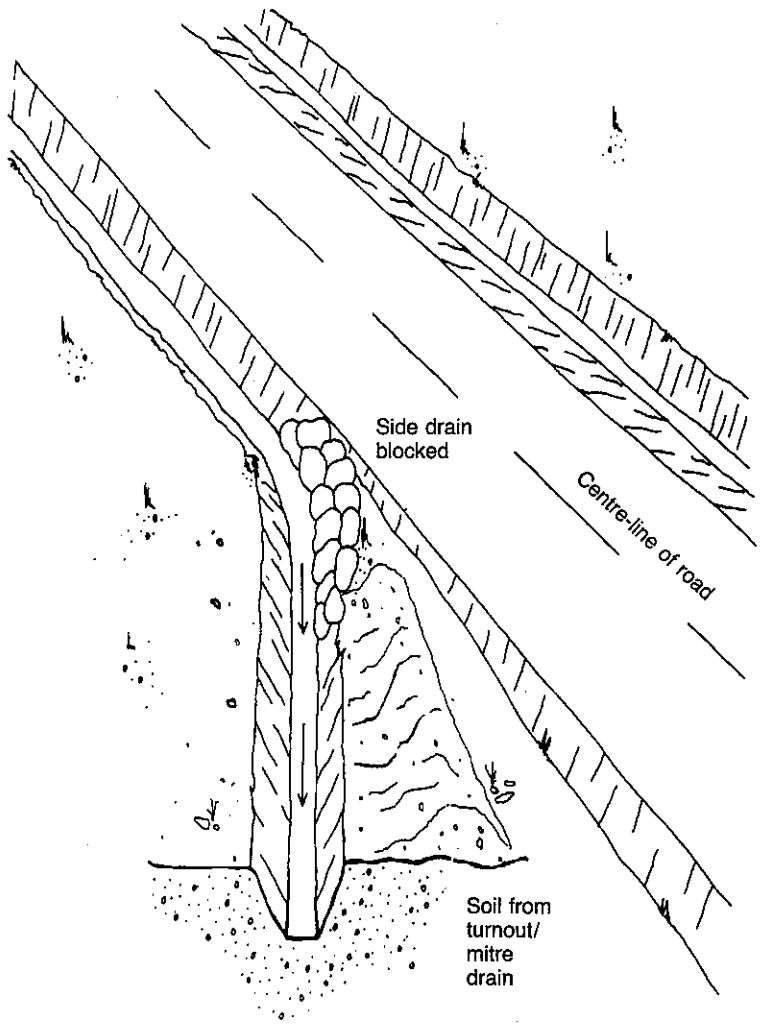


Figure 11.1 Turnouts (copyright International Labour Organisation) (ref. 1).

arid soils. Intercepting ditches may be built well back (3 m) from the top of the cut slope and generally on a flat grade until the water can be spread or emptied into a natural watercourse.

Hydraulic calculation

The hydraulic capacity of drainage channels is often designed to contain a 5- or 10-year frequency storm runoff. The estimated runoff for the two-year frequency storm can be used for determining the needs, type, and dimensions of special channel linings for erosion control.

The design discharge is calculated according to the rational formula, as in Chapter 10, and the capacity of an open channel is calculated according to the Manning equation which gives a reliable estimate of uniform flow conditions:

$$Q = A \times v = A \times \frac{1}{n} \times R^{2/3} \times I^{1/2}$$

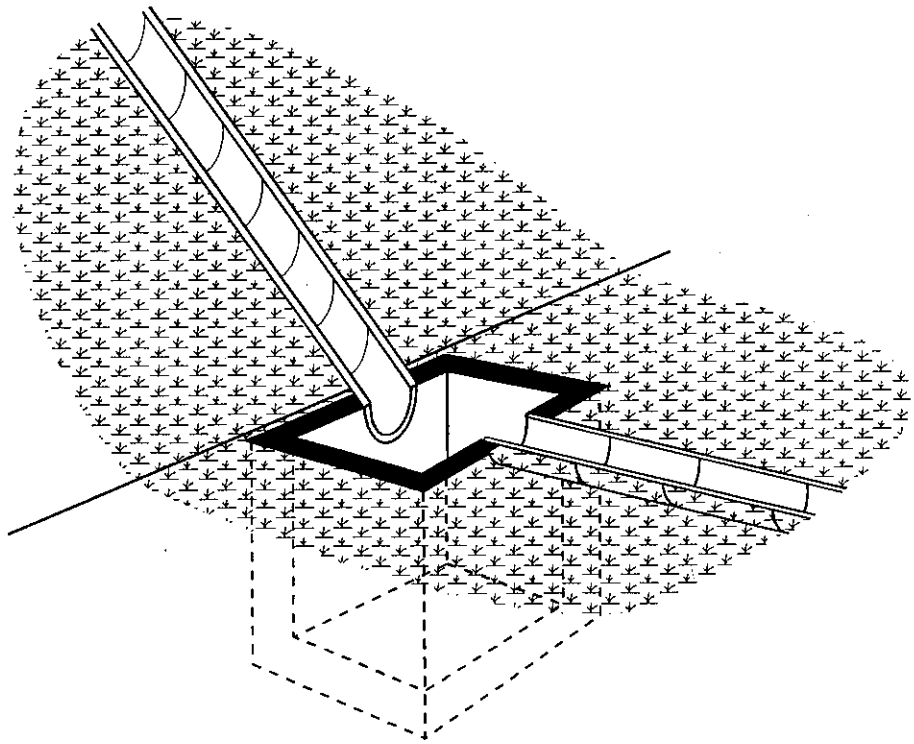


Figure 11.2 Chute outlet.

where:

Q = capacity in $m^3 sec^{-1}$;
A = channel cross-sectional area in m^2;
v = mean velocity in $m sec^{-1}$;
n = Manning roughness coefficient;
R = hydraulic radius A/P in m;
I = slope in $m m^{-1}$;
P = wetted perimeter in m.

Table 11.1 indicates some Manning roughness coefficients at different water depths.

11.3 CROSS-DRAINAGE

Road alignment and drainage

Cross-drainage structures can be very costly and it is therefore important to analyse all major cross-drainage along the road alignment before final selection of a new road alignment.

Where there is a choice in the selection of the position of a stream crossing, it is desirable that, as far as possible, the stream is located:

Table 11.1 Manning roughness coefficient (ref. 2).

	Water depth		
Type of lining	*0–15 cm*	*15–60 cm*	*>60 cm*
Concrete	0.015	0.013	0.013
Grouted riprap	0.040	0.030	0.028
Stone masonry	0.042	0.032	0.020
Soil cement	0.025	0.022	0.020
Asphalt	0.018	0.016	0.016
Bare soil	0.023	0.020	0.020
Rock cut	0.045	0.035	0.025
Riprap (2.5 cm)	0.044	0.033	0.030
Riprap (5 cm)	0.066	0.041	0.034
Riprap (15 cm)	0.104	0.069	0.035
Riprap (30 cm)	–	0.078	0.040

- on a straight reach of the stream, away from bends;
- as far as possible from the influence of large tributaries;
- on a reach with well-defined banks;
- at a site which makes straight approach roads feasible;
- at a site which makes a right-angle crossing possible.

To determine the type of cross-drainage relevant information on hydrology must be collected, and predictions about the level of traffic should be made.

Type of structure

The following types of structure may be considered:

- fords
- drifts
- culverts
- bridges.

Fords

The simplest river crossing is a ford. This utilizes a suitable existing river-bed and is appropriate for shallow, slow moving watercourses with little probability of flash floods. The traffic volumes may be up to about 100 vehicles per day.

Gravel or stones can be used to line the bottom of the ford to provide a firm footing for vehicles. Fords should normally only be used for rivers that do not flood as this may cause the ford to be washed away. However, repair or replacement is cheap.

Drifts

If the river-bed is not able to carry vehicles, a concrete slab may be constructed in the bed. The crossing is then termed a drift or an Irish crossing (Figure 11.3). A drift is suitable as a crossing for rivers that are normally fordable but prone to floods. Drifts are able to carry the same traffic volumes as fords.

Vented drift

Where the river is running most of the year, the drift can be provided with openings to permit water to pass below road level and reduce the frequency and depth of overtopping during flash floods. This type of crossing is called a vented drift or a causeway.

Culverts

Culverts are used to convey water from streams below the road and to carry water from one side-ditch to the other. The difference between culverts and

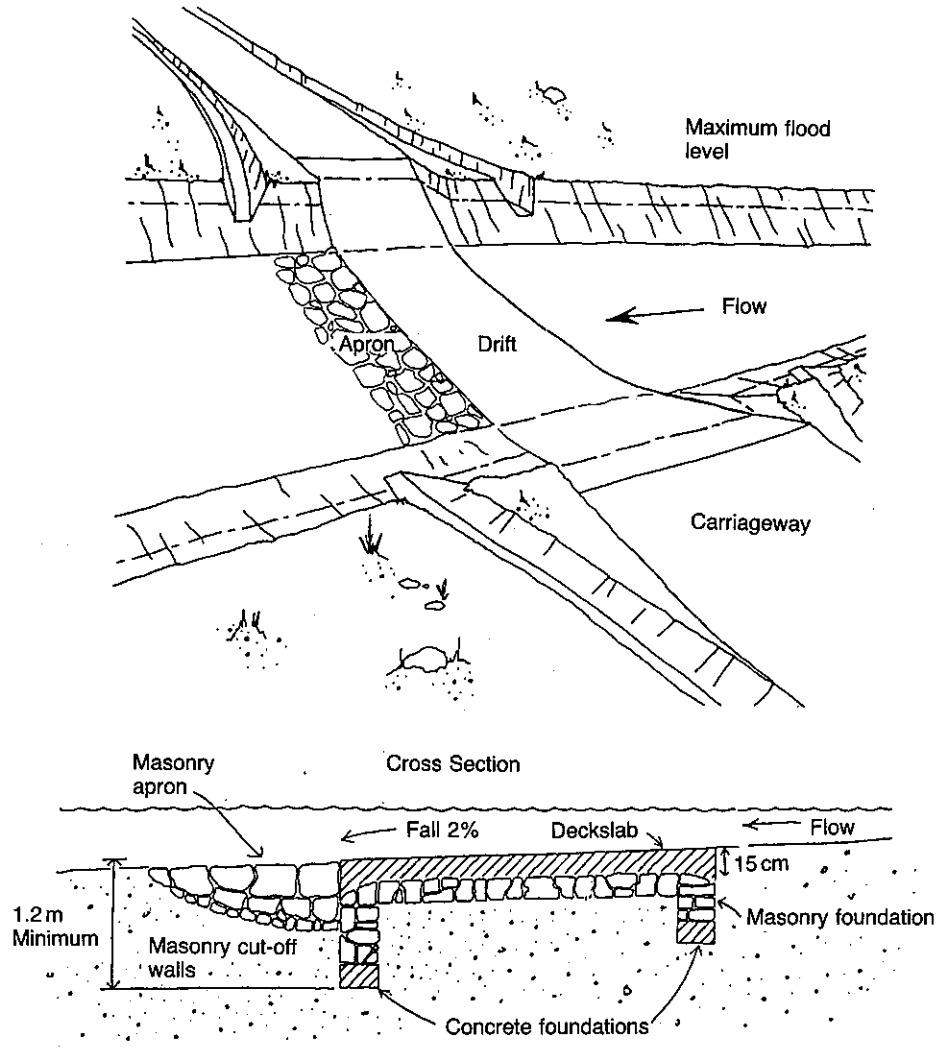


Figure 11.3 Drift (copyright International Labour Organisation) (ref. 1).

bridges is that culverts are placed in the embankment below the road pavement whereas bridge decks form part of the pavement. Usually bridges have a larger span than culverts and bridges may be constructed to allow for the passage of boats, whereas culverts are frequently designed with the cross-section flowing full.

Culverts usually consist of a concrete or steel pipe (Figure 11.4), or a reinforced concrete box. Most countries make precast concrete pipes of up to one metre diameter and these may be cost-effective, provided that they can be transported and handled. Corrugated galvanized steel pipes, often known by the trade name 'Armco', are available in large diameters and are lighter and easier to handle. There should be little maintenance required for either material other than an annual inspection and clearing of accumulated silt or debris, although corrosion may occur to metal pipes in some circumstances.

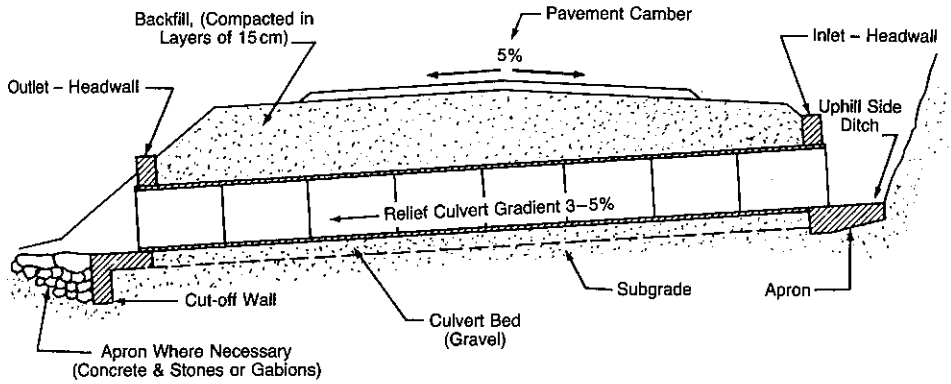


Figure 11.4 Pipe culvert (ref. 4).

Most culverts begin upstream with headwalls and terminate downstream with endwalls. Headwalls direct the flow into the culvert, while endwalls provide a transition from the culvert to the outlet channel. Both protect the embankment from erosion by flood waters. Headwalls and endwalls are sometimes together called headwalls. Straight headwalls placed parallel to the roadway are used mainly with smaller pipe culverts. For larger culverts the headwalls are normally supplemented with wingwalls at an angle to the embankment (Figure 11.5). Inlets and outlets

Most head- and wingwalls are made of concrete and cast in place, although masonry is also used. In all cases, a cut-off wall (toe-wall) extending below the level of expected scour should be incorporated in the design of the outlet. Often a paved apron extending beyond the cut-off wall is a wise addition. The culvert inlet may also be provided with an apron (Figure 11.4).

For larger volumes of water one can use several pipes in parallel under the road. Multiple pipes can also be used where the planned embankment height is insufficient to cover a single pipe of sufficient diameter. However, pipes of less than a metre diameter are not recommended since they are difficult to maintain. Multiple culverts

Reinforced concrete box culverts may also be used either singly or in parallel in the case of relatively large volumes of water. These culverts are normally cast in place, although smaller sizes may be precast. The design of culverts is discussed in the next section.

Bridges are required for crossing of streams and rivers where culverts would provide insufficient capacity, or where the road crosses an obstruction such as a railway or canal. There is no restriction to traffic unless the width of the structure is less than the road width. The design of bridges is outside the scope of this book. Bridges

11.4 CULVERT DESIGN

11.4.1 Basis

The design discharge used in culvert design is usually estimated on the basis of a preselected recurrence interval (Chapter 10). The culvert is designed to operate in a manner that is within acceptable limits of risk at that flow rate. Design discharge

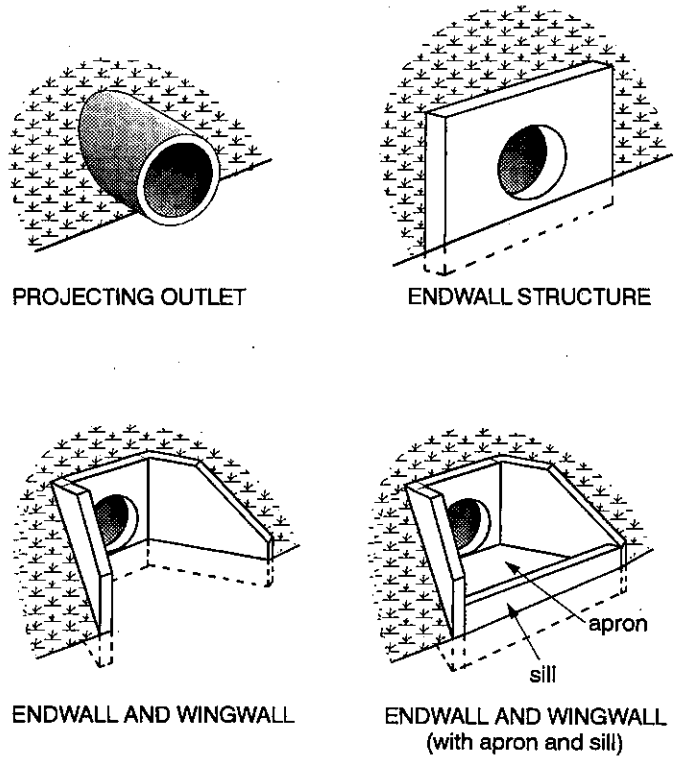


Figure 11.5 Types of culvert outlets.

Location

Location of the culverts should be selected very carefully. The alignment of the culvert should generally conform to the alignment of the natural stream. The culvert should, if possible, cross the roadway at a right angle in the interests of economy. However, skew culverts located at an angle to the centre-line of the road are needed in many instances.

The slope of the culvert should generally conform to the existing slope of the stream. To avoid silting, the slope of the culvert should not be less than 1%.

Type of inlet

For culverts flowing under inlet control (discussed in the following sections), the culvert capacity is affected by the inlet configuration. Wingwalls improve the hydraulic characteristics. By rounding the entrances of circular pipes or tapering the inlets of rectangular or square cross-sections, the flow accommodated at a given head can be substantially increased.

Definitions

Before explaining the procedure for determination of culvert size it is necessary to define the following concepts:

- headwater depth;
- tailwater depth;
- outlet velocity;
- culvert flow with inlet control;
- culvert flow with outlet control.

Headwater

Culverts generally constrict the natural flow of a stream and cause a rise in the

upstream water surface. The elevation of this water surface at the culvert entrance is termed headwater elevation, and the total flow depth in the stream measured from the culvert inlet invert is termed headwater depth. The headwater elevation should not be too high. High headwater may damage the culvert and the roadway and thereby interrupt the traffic. High headwater may also damage upstream property and cause hazard to human life.

The headwater elevation for the design discharge should be at least 0.5 m below the edge of shoulder elevation. A ratio between headwater depth and height of culvert opening (HW/*D*) equal to 1.2 is recommended in cases where insufficient data are available to predict the flooding effect from high headwater. Headwater depth is a function of the discharge, the culvert size, and the inlet configuration.

The designer should verify that the drainage divides (watershed) are higher than the design headwater elevations. In flat terrain drainage divides are often undefined or non-existent and culverts should be located and designed for least disruption of the existing flow distribution.

Tailwater

Tailwater depth is the flow depth in the downstream channel measured from the invert at the culvert outlet. Tailwater depth can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

An approximation of the tailwater depth, TW, can be made by use of the Manning equation (Chapter 10) if the outlet channel is reasonably uniform in cross-section, slope and roughness. However, tailwater conditions during floods are sometimes controlled by downstream obstruction or by water stages in another stream. Much experience is then needed to evaluate tailwater depth. A field inspection should always be made to check on features that may influence tailwater conditions.

Outlet velocity

The outlet velocity of highway culverts is the velocity measured at the downstream end of the culvert, and it is usually higher than the maximum natural stream velocity. This higher velocity can cause stream-bed scour and bank erosion for a limited distance downstream from the culvert outlet.

Culvert flow

There are two major types of culvert flow:

- flow with inlet control;
- flow with outlet control.

For each type of control, a different combination of factors are controlling the hydraulic capacity of a culvert. The determination of actual flow conditions can be difficult. Therefore, the designer should check for both types of flow and design for the most adverse condition.

Inlet control

A culvert operates with inlet control when the flow capacity is controlled at the entrance by the following factors:

- culvert type (shape of barrel);
- type of culvert inlet;
- culvert cross-sectional area;
- headwater depth.

When a given culvert operates under inlet control the headwater depth determines the culvert capacity with the barrel usually flowing only partially full. Figure

11.6 shows sketches illustrating the inlet control flow for unsubmerged and submerged entrances.

Outlet control

When a culvert operates under outlet control the flow capacity is determined by the same factors as under inlet control, but in addition to that the performance depends on:

- roughness of the inner surface of the culvert (barrel roughness);
- longitudinal slope of the culvert (barrel slope);
- tailwater depth or critical depth.

Culverts operating under outlet control may flow full or partly full depending on various combinations of the determining factors. Typical outlet control flow situations are shown in Figure 11.7.

11.4.2 Design procedure

In order to find the size of culvert and the velocity at the outlet under a given set of conditions it is first necessary to determine the probable type of flow under which the culvert will operate. Determination of probable type of flow is possible by comprehensive hydraulic computations. However, the need for making such computations may be avoided by computing the headwater depth for both inlet control and outlet control and then using the highest value to indicate the type of control.

The design procedure may be divided into six steps.

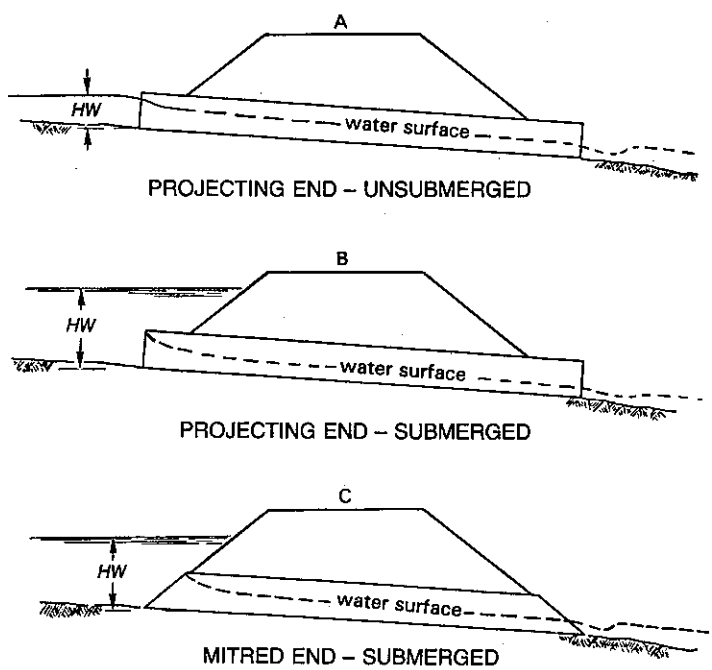


Figure 11.6 Inlet control (Transportation Research Board, ref. 3).

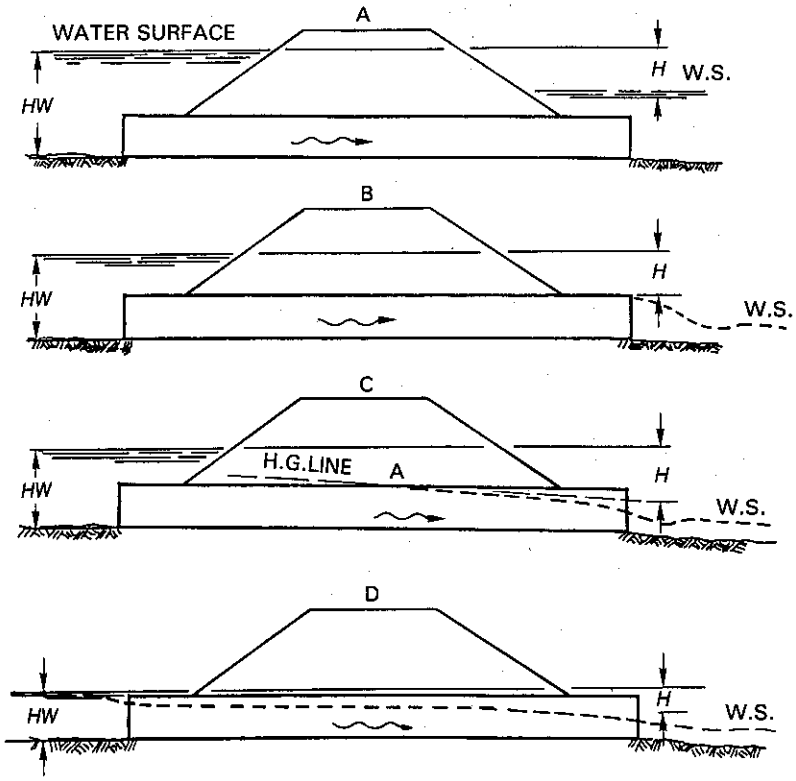


Figure 11.7 Outlet control (Transportation Research Board, ref. 3).

List the following design data: Step 1

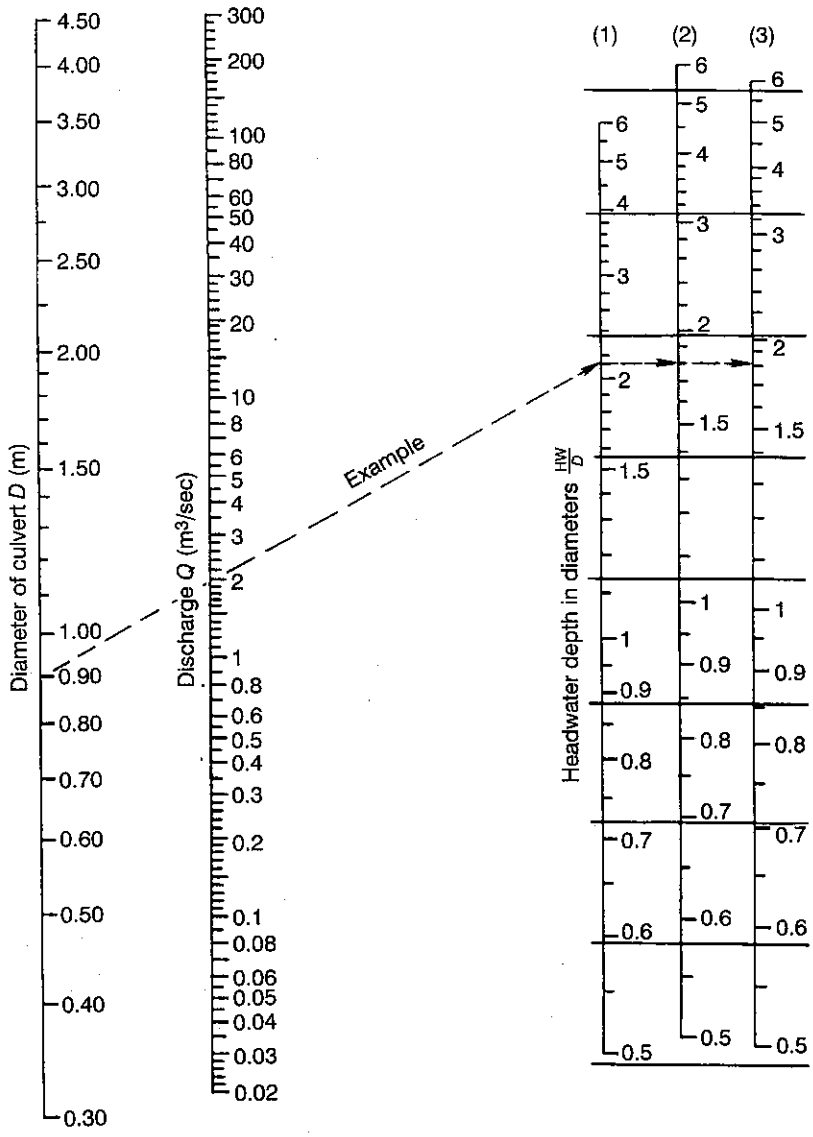
(a) design discharge in $m^3 sec^{-1}$;
(b) approximate length of culvert in m;
(c) slope of culvert in $m\,m^{-1}$;
(d) allowable headwater depth in m;
(e) mean and maximum flood velocity in natural stream in $m\,sec^{-1}$;
(f) type of culvert for the first trial, including barrel material, barrel cross-sectional shape and inlet type.

Select the first trial size of culvert. Step 2
Find the headwater depth for the trial-size culvert: Step 3

(a) Assuming inlet control, find the headwater depth, HW, by using an appropriate nomograph. Figure 11.8 shows an inlet-control nomograph for concrete pipe culverts and three different inlet types. Tailwater conditions are to be neglected in this determination.

If HW is greater or less than allowable, try another size of culvert until HW is acceptable for inlet control.

(b) Assuming outlet control, approximate the tailwater depth, TW, during flood in the outlet channel.

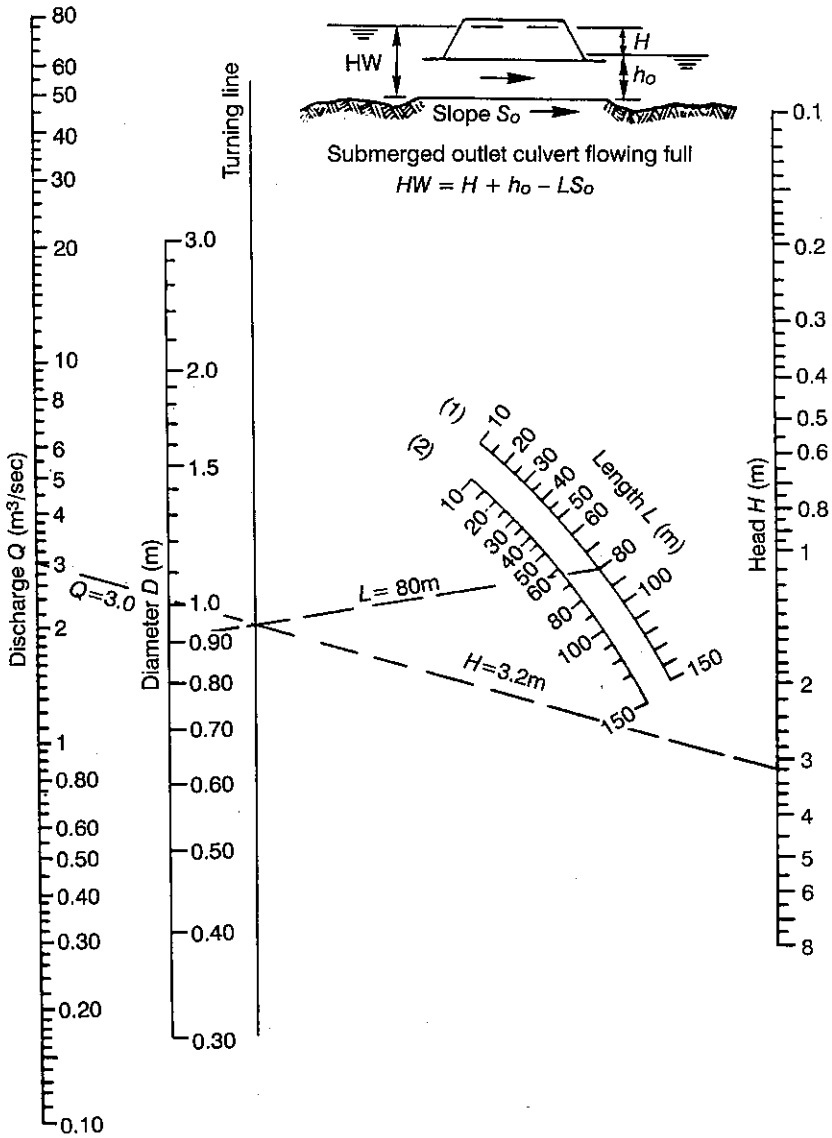


Scale (1): Inlet with square edge and headwall.
Scale (2): Inlet with groove end (socket end) and headwall.
Scale (3): Inlet with groove end (socket end) projecting.

Figure 11.8 Inlet-control nomograph for concrete pipe culverts (after Transportation Research Board, ref. 3).

For tailwater elevation equal to or greater than the top of the culvert at the outlet set, h_0 equal to TW and find HW by the following equation:

$$\text{HW} = H + h_0 - LS_0$$



Scale (1): Inlet with groove end (socket end) projecting, and inlet with groove end (socket end) and headwall.
Scale (2): Inlet with square edge and headwall.

Figure 11.9 Outlet-control nomograph for concrete pipe culverts flowing full (after Transportation Research Board, ref. 3).

where:

HW = tailwater depth in m;
H = head loss in m determined from appropriate nomograph; Figure 11.9 shows an outlet-control nomograph for concrete pipe culverts flowing full;

h_0 = vertical distance in m from invert at outlet to the hydraulic grade line (in this case h_0 equals TW);
S_0 = slope of barrel in $m\,m^{-1}$;
L = culvert length in m.

For tailwater elevations less than the top of the culvert at the outlet, find the headwater depth, HW, as above except that:

$$h_0 = \frac{d_c + D}{2}$$

or TW, whichever is the greater,

where:

d_c = critical depth in m; see Figure 11.10 for pipe culverts;
D = height of culvert opening in m (for pipe culverts D equals the diameter).

(c) Compare the headwaters found for (a) inlet control and (b) outlet control. The highest headwater governs and indicates the flow control existing under the given conditions for the trial size selected.
(d) If outlet control governs and the HW is higher than is acceptable, select a larger trial-size culvert and find the new HW for outlet control. Inlet control need not be checked, as the smaller size was satisfactory for this control.

Step 4 Try a culvert of another type or shape and determine the size and HW by the above procedure.

Step 5 Compute the outlet velocity for the size and types of culverts to be considered and determine the need for channel protection:

(a) If inlet control governs, outlet velocity can be assumed to equal mean velocity in open-channel flow in the culvert as computed by the Manning equation for the rate of flow, cross-sectional area, barrel roughness, and barrel slope of culvert selected.
(b) If outlet control governs, outlet velocity equals Q/A_0, where Q is the rate of flow and A_0 is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel use A_0 corresponding to d_c or TW, whichever gives the greater area of flow. A_0 should not exceed the total cross-sectional area of the culvert barrel.

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

A form that may be used when designing pipe culverts is shown in Figure 11.11.

11.5 EROSION AND SCOUR PROTECTION

11.5.1 Ditches

Permissible velocity

Usually ditches are unlined, and often suffer from scour. Tables 11.2 and 11.3 indicate maximum permissible velocities (V_p) that could be used without scour for maximum water depths of 1 metre.

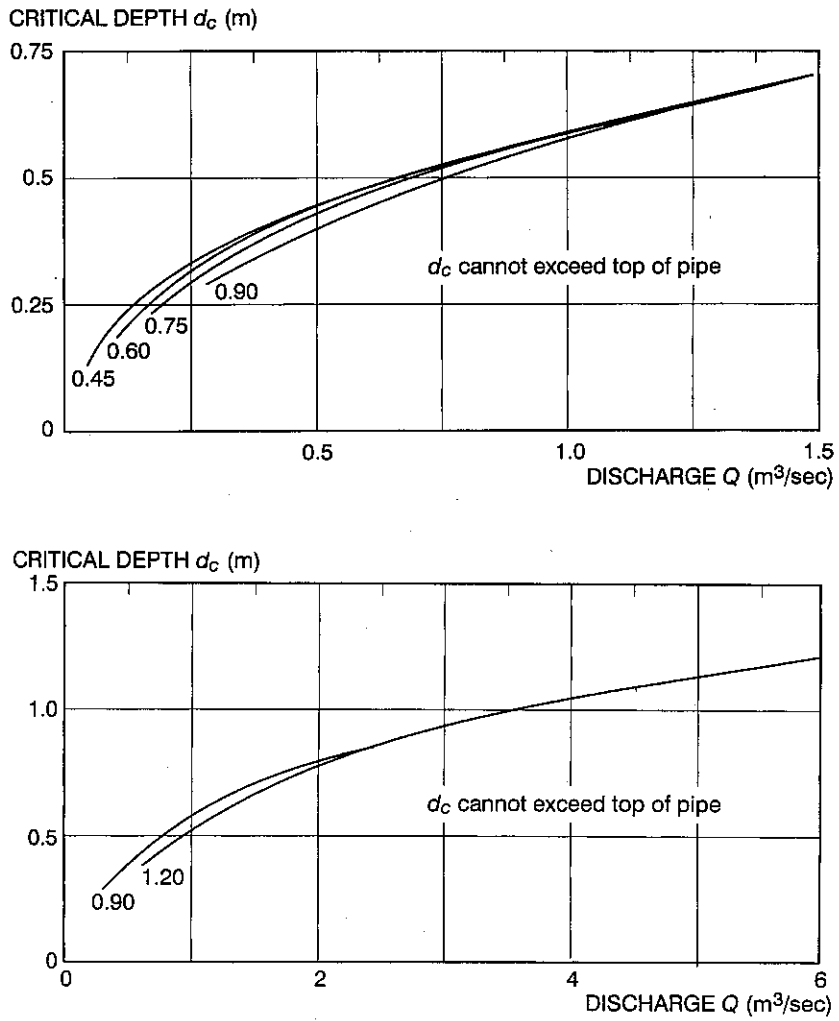


Figure 11.10 Critical depth for pipe culverts with different diameters in m (after Transportation Research Board, ref. 3).

Erosion control

The amount of erosion control and maintenance can be minimized to a great extent by using:

- flat side slopes;
- turnouts and intercepting ditches;
- ditch checks or drop structures (Figure 11.12);
- asphalt or concrete kerbs with chutes on the road section with high embankments;
- protective lining of ditches by rigid linings such as cast-in-place concrete, stone masonry or grouted riprap;
- protective covering of ditches by flexible linings such as riprap, wire-enclosed riprap, vegetation or synthetic material.

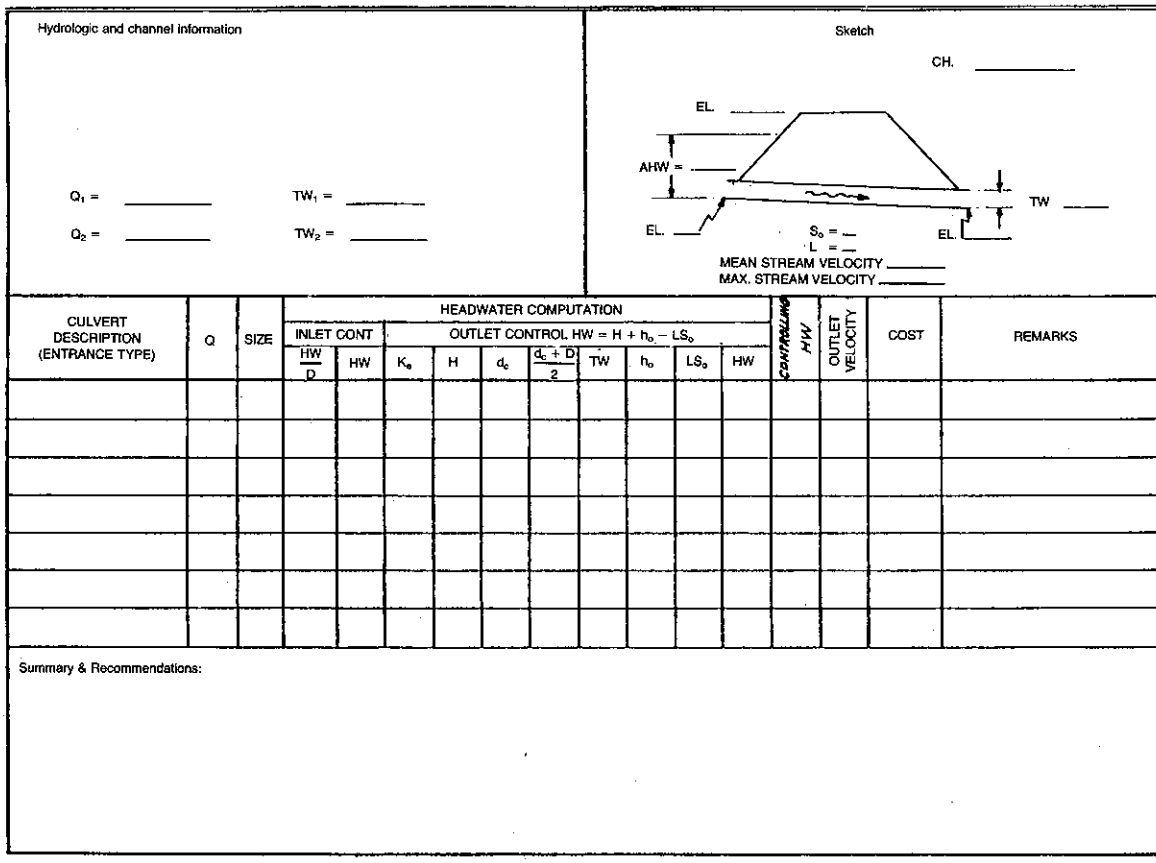


Figure 11.11 Design form for pipe culverts (Transportation Research Board, ref. 3).

Table 11.2 Permissible velocities for ditches lined with various grass covers (ref. 5).

Type of lining	*Permissible velocity* ($m\,sec^{-1}$)
Well-established grass on any good soil	1.8
Meadow type grass, e.g. bluegrass	1.5
Bunch grasses, exposed soil between plants	1.1
Grains, stiff stemmed grasses that do not bend over under shallow flow	0.9

Rigid linings

Rigid linings are useful in flow zones where high shear stress or non-uniform flow conditions exist, such as at transitions in ditch shape or at an energy dissipation structure. In areas where loss of water or seepage from the ditch is undesirable, they provide an impermeable lining. Cast-in-place concrete or masonry linings often break up and deteriorate if foundation conditions are poor. Once a rigid lining deteriorates, it is very susceptible to erosion.

Flexible linings

Flexible linings made of riprap or vegetation are suitable linings for hydraulic

Table 11.3 Permissible velocities in excavated ditches (ref. 5).

Soil type or lining (no vegetation)	*Permissible velocity ($m\,sec^{-1}$)*		
	Clear water	*Water-carrying fine silts*	*Water-carrying sand and gravel*
Fine sand (non-colloidal)	0.45	0.75	0.45
Sandy loam (non-colloidal)	0.55	0.75	0.60
Silt loam (non-colloidal)	0.60	0.90	0.60
Ordinary firm loam	0.75	1.05	0.70
Volcanic ash	0.75	1.05	0.60
Fine gravel	0.75	1.50	1.15
Stiff clay (very colloidal)	1.15	1.50	0.90
Graded, loam to cobbles (non-colloidal)	1.15	1.50	1.50
Graded, silt to cobbles (colloidal)	1.20	1.70	1.50
Alluvial silts (non-colloidal)	0.60	1.05	0.60
Alluvial silts (colloidal)	1.15	1.50	0.90
Coarse gravel (non-colloidal)	1.20	1.85	2.00
Cobbles and shingles	1.50	1.70	2.00
Shales and hard pans	1.85	1.85	1.50
Rock	Negligible scour at all velocities		

conditions similar to those requiring rigid linings. Because flexible linings are permeable, they may require protection of underlying soil to prevent washout. For example, filter cloth is often used with riprap to inhibit soil piping.

Vegetative lining is suited to hydraulic conditions where uniform flow exists and shear stresses are moderate. Vegetative channel linings are not suited to continued flow conditions or long periods of submergence.

11.5.2 Culverts

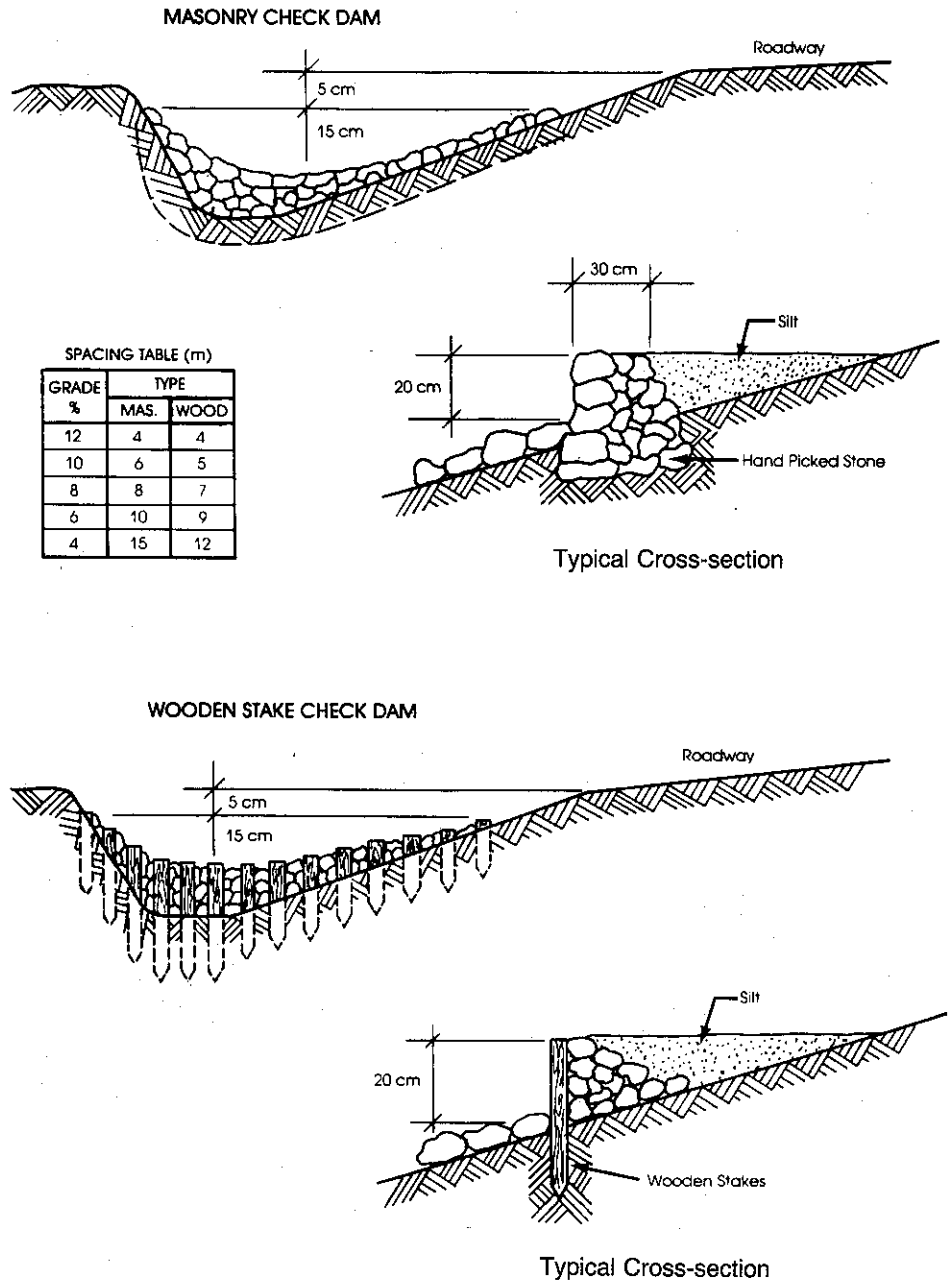
Unchecked erosion is the prime cause of culvert failure. The greatest scour potential is at the culvert outlet where high velocities may necessitate scour protection or energy dissipation.

Inlet

At culvert inlets erosion from vortexes and flow over wingwalls is generally not a major problem. However, erosion may be caused by high inlet flow velocity.

Velocity near the inlet may be approximated by dividing the flow rate by the area of the culvert opening. The risk of channel erosion should be judged on the basis of this approach velocity. Based on these considerations the extent of the slope protection could be limited to cover an area in the immediate proximity of the culvert entrance. However, where the flow enters the transition between the embankment and the wingwalls, high local velocities and flow disturbances are expected, justifying the slope protection to be extended a certain distance beyond the wingwalls.

In cases where it is impossible to locate the culvert in the same direction as that of the stream, e.g. culverts carrying water below the road from one side ditch to the other, water may flow along the embankment before entering the culvert. As runoff begins, the water will flow along the embankment with a relatively small depth, but not necessarily with a low velocity, until reaching the culvert entrance. Care should therefore be taken to protect the embankment upstream of the culvert from erosion.



SPACING TABLE (m)

GRADE %	TYPE	
	MAS.	WOOD
12	4	4
10	6	5
8	8	7
6	10	9
4	15	12

Figure 11.12 Ditch checks (ref. 4).

If flow velocity near the inlet indicates a possibility of scour threatening the stability of wingwall footings, erosion protection should be provided. A concrete apron with cut-off wall between wingwalls is the most satisfactory means for providing this protection.

Most of the inlet failures reported have occurred on large flexible-type pipe culverts with projected entrances or with mitred entrances (entrance conforming with the embankment slope) without headwalls and wingwalls or other entrance protection.

Outlet

Erosion at culvert outlets is a common problem. Determination of the flow condition, scour potential, and channel erodibility at the outlet should be standard procedure in the design of all road culverts.

The only safe procedure is to design on the basis that erosion at a culvert outlet and downstream channel is to be expected. A reasonable procedure is to provide at least minimum protection, and then inspect the outlet channel after major rainfalls to determine if the protection must be increased or extended.

Type of scour

Two types of scour can occur in the vicinity of culvert outlets – local scour and general channel degradation.

Local scour

Local scour is the result of high-velocity flow at the culvert outlet, but its effect extends only a limited distance downstream. Natural channel velocities are almost universally less than culvert outlet velocities, because the channel cross-section including its flood plain is generally larger than the culvert flow area.

Channel degradation

Channel degradation may proceed in a fairly uniform manner over a long length, or may be evident in one or more abrupt drops progressing upstream with every runoff event. The latter type, referred to as head-cutting, can be detected by periodic maintenance inspections following construction.

The highest velocities will be produced by long, smooth-barrel culverts on steep slopes. These cases will no doubt require protection of the outlet channel at most sites. However, protection is also often required for culverts on mild slopes.

Scour protection

To prevent scour at outlet, energy dissipators such as drop structures, riprap basin, stilling basin, etc. are the most far-reaching anti-erosion devices.

Practice

Standard practice is often to use the same treatment at the culvert entrance and exit. It is important to recognize that the inlet is designed to improve culvert capacity or reduce headloss while the outlet structure should provide a smooth flow transition back to the natural channel.

11.5.3 Bridges

Three principal forms of scour are relevant to the design of major drainage structures, particularly bridges. Those forms are the natural scour, the general scour, and the local scour.

Definitions

Natural scour occurs in natural streams, with the migration of bed forms, shifting of the direction of flow, and at bends and contractions. General scour occurs due to the introduction of a constriction in a stream channel such as a bridge crossing, as a result of increased velocities in the contracted section. General scour is avoided by designing bridges with a sufficiently large span.

Local scour occurs in more concentrated locations, as for instance around bridge piers and embankment ends because of high local velocities and flow disturbances such as eddies and vortices. The effects of natural, general and local scour are generally additive where they occur at the same location.

The influence of natural and general scour has an impact on bridge design in two ways. When estimating the highest water level during the flood peak, a scouring effect will tend to decrease this level, thus diminishing the risk of

overtopping of the structure. On the other hand, natural and general scour in conjunction with local scour can increase the risk of undercutting the bridge foundations.

Great difficulties, theoretical as well as practical, are encountered when the depth of the scour has to be estimated. When such estimates are used for design purposes a large safety factor will have to be applied. The total effect of natural, general and local scour is often estimated by calculating only the effect of local scour and using a large safety factor.

Local scour

Bridge piers built in alluvial material, unless set deeply, or provided with a local protection, are often exposed to undercutting and may settle or collapse. The depth of local scour that may be expected in any given location depends upon the duration and peak-flow of the flood, the susceptibility of the bed material to rapid erosion, the depth of flow, the mean flow velocity and the pier size. Even when field measurements taken during a flood happen to be available, estimation of scour depth is difficult.

Several methods for estimating the scour depth for local scour around bridge piers have been suggested during the past 30–40 years. According to ref. (6) the best estimate of maximum scour depth around circular piers is obtained from the following equations:

1. For clear-water scour, where bed material is removed from the scour hole and not replaced:

$$\frac{d_s}{b} = 1.84 \times F_c^{0.25} \times \left(\frac{y}{b}\right)^{0.3}. \tag{1}$$

2. For scour that occurs with general sediment transport:

$$\frac{d_s}{b} = 2.0 \times (F - F_c)^{0.25} \times \left(\frac{y}{b}\right)^{0.5}, \quad (F - F_c \geqslant 0.2) \tag{2}$$

where:

d_s = scour depth in m;
y = mean depth of approach flow in m;
b = width of pier in m;
F = Froude number;
F_c = critical Froude number for beginning sediment motion.

The Froude numbers are calculated in four steps:

Step 1

Find the boundary Reynolds number from:

$$R = \frac{\sqrt{gyI}}{v} \times D_{50}$$

where:

R = boundary Reynolds number;
g = acceleration of gravity (9.81 m sec^{-2});
I = hydraulic gradient or river slope in m m^{-1};
v = kinematic viscosity of water (10^{-6} m^2 sec^{-1} at 20 °C);
D_{50} = mean size of bed material in m.

Read the dimensionless critical bed shear stress θ_c from Shield's diagram, Figure 11.13. Step 2

Calculate the critical velocity from: Step 3

$$V_c^2 = \theta_c \times \left(6 + 2.5 \ln \frac{y}{2.5 D_{50}}\right)^2 \times \left(\frac{\gamma_s}{\gamma} - 1\right) \times g \times D_{50}$$

where:

V_c = critical velocity in $\mathrm{m\,sec^{-1}}$;
θ_c = dimensionless critical bed shear stress;
γ_s = specific weight of sediment grains $\mathrm{N\,m^{-3}}$;
γ = specific weight of water $\mathrm{N\,m^{-3}}$.

Calculate the Froude numbers from: Step 4

$$F = \frac{V}{\sqrt{g \times y}}, \qquad F_c = \frac{V_c}{\sqrt{g \times y}}$$

where V is the mean velocity of approach flow in $\mathrm{m\,sec^{-1}}$.

The scour depth d_s is determined as the largest of the two values calculated by Equation (1) and Equation (2). d_s

Protection works such as rock aprons will limit the depth of scour. Rock aprons should at least be laid below the general scour level. River banks can be protected by rock aprons as well as by other revetment such as gabions and precast concrete blocks. At bridge crossings guide banks can help control flow through the bridge openings and control erosion at river banks. Protection

The best solution for minimizing scour and flood damage at new bridges would be to: New bridges

- locate the bridge in a way that adverse flood flow patterns are avoided;
- streamline bridge elements to minimize obstructions to the flow;
- deepen the foundations to accommodate scour.

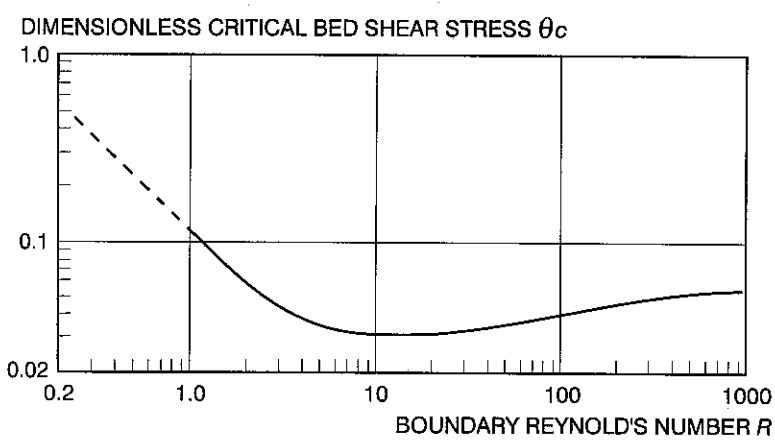


Figure 11.13 Shield's diagram (reproduced by permission of American Society of Civil Engineers, ref. 7).

Existing bridges

For existing bridges the alternatives available to protect the bridge from scour and flood damage are listed below in a rough order of cost:

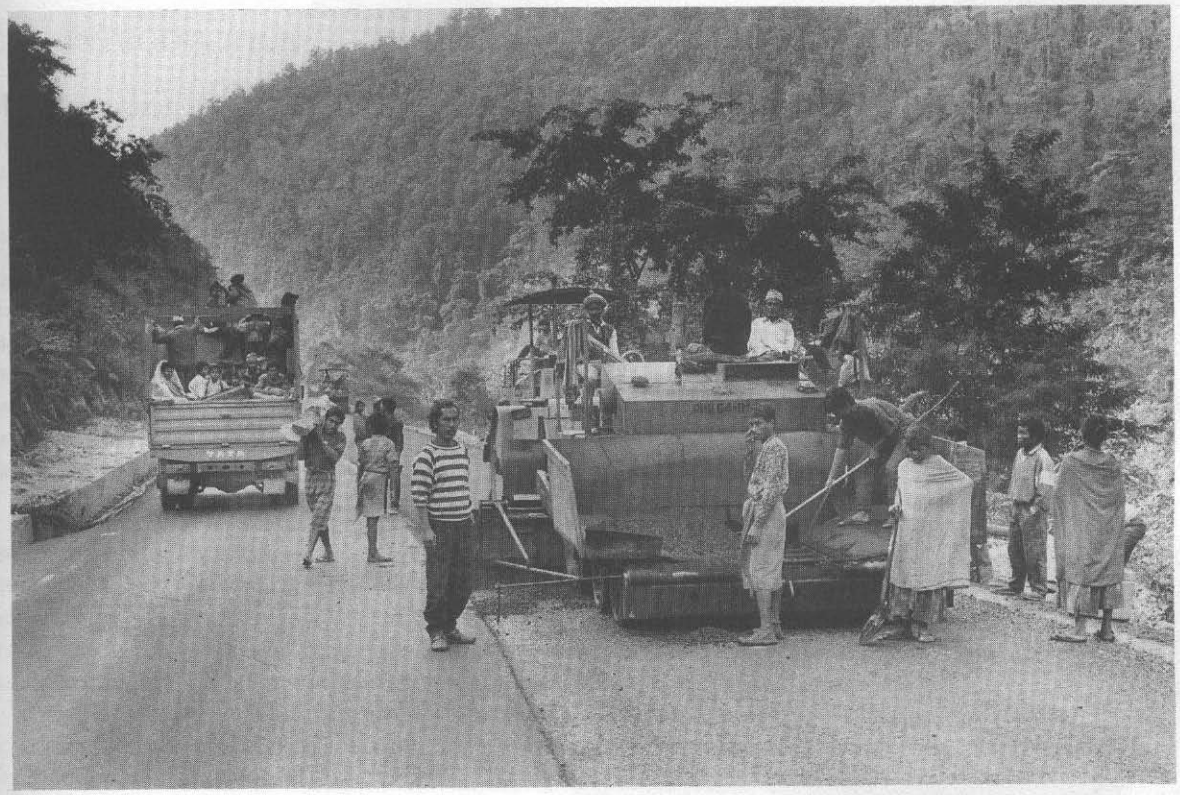
- providing riprap at piers and abutments;
- constructing guide banks;
- straightening out and widening of the stream;
- strengthening the bridge foundations;
- constructing energy dissipation devices such as sills or drop structures;
- constructing relief bridges, or lengthening existing bridges;
- constructing a floodway (a road section where flood water can pass over the road) in the vicinity of the bridge by lowering the longitudinal profile of the road.

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PART 6

Pavements



Asphalt paving in Nepal. (Photo by Jorgen Schytte)

12 Soil investigation

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12.1 INTRODUCTION

Purpose

A soil investigation is an integral part of the location, design and construction of highways. Soil conditions as well as topography, land use, environment problems and political considerations must be weighed in selecting the position of the road.

The soil investigation is to provide pertinent information about soil and rock for a decision on one or more of the following subjects:

- selection of roadway alignment;
- decision on the need for subgrade or embankment foundation treatment;
- investigation of slope stability in cuts and embankments;
- location and design of ditches and culverts;
- selection and design of the roadway pavement;
- location and evaluation of suitable borrow and construction materials;
- design of foundations for bridges and other structures.

Approach

In selecting the alignment for a new highway the first step is normally to define a number of conceivable corridors between the end termini of the road. The next step is to select the best corridor for the proposed road and define within it one or more different alignments. These alignments are compared, and a final selection is made for design purposes. The process involves continuous searching and selecting, using increasingly more detailed knowledge at each decision-making stage.

Preliminary investigation

An early phase of the soil investigation (the preliminary soil investigation) encompasses collection and examination of all existing information. This may include the identification of soil types from topographic maps, geological maps, soil maps, aerial photos and satellite images, registration of groundwater conditions, and examination of existing roadway cuts. The visual examination of existing maps, etc., may be coupled with a small amount of sampling and testing. The preliminary soil investigation will aid in securing broad understanding of soil conditions and associated engineering problems that may be encountered. Furthermore, the information is of great value in planning and conducting the detailed soil investigation that is necessary for design and construction of the road.

Detailed investigation

The detailed soil investigations may be divided into field investigations and laboratory testing. The field investigations include geophysical explorations, making

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of test pits and borings, sampling of soils and rocks, registration of soil profiles and measurements of groundwater levels. Laboratory testing includes testing on representative samples.

12.2 EVALUATION OF EXISTING INFORMATION

12.2.1 Interpretation of existing maps

Topographic maps

Most countries in the world are covered by topographic maps on the scale of 1:50 000 to 1:250 000. These maps may be used as an aid to geological interpretation, to identify drainage networks, and to estimate gradients and earthwork volumes. However, topographic maps may be inaccurate, and they are often outdated.

Geological maps

Most countries are covered by national geological maps of scale 1:100 000 or smaller. More detailed mapping may exist, but few developing countries are covered by large-scale maps.

Geological maps normally depict the bedrock up to the level beneath the soil. In some cases rock types can be correlated with particular soil types, but for the road engineer the main use of geological maps is for planning purposes and for providing background information for the interpretation of aerial photos.

Soil maps

Soil maps are mainly produced for agricultural purposes. Only limited areas in the developing countries are covered by soil maps. Engineering particulars cannot be read from agricultural soil maps, but they are useful for planning purposes, because they indicate where variations in the soil types can be expected.

12.2.2 Air photo interpretation

Air photos

Air photos are normally stereoscopic photographs taken in preparation for land mapping. The scale is typically 1:20 000 or 1:50 000. Many developing countries have complete air photo cover, and most countries have at least 50% cover. Prints of the photos are kept by the local survey department or a user department.

Terrain information visible on air photos can be used for identification of most of the common bedrock types, associated residual soils, transported soils, and organic soils but detailed evaluation of ground conditions should never be made by photo interpretation alone.

A standard text on air photo interpretation is ref. (1). Air photo interpretation for terrain evaluation is based on a systematic observation and evaluation of key elements that are studied stereoscopically. These are:

- topography
- drainage pattern
- erosion
- photo tone
- vegetation.

Air photo interpretation is best learned through experience, and the following examples are only presented to illustrate the vast potentialities of the technique.

Topography

Bold, massive, relatively flat-topped hills with steep hillsides indicate that the bedrock is horizontally bedded sandstone. In an arid climate, closely cut up terrain

with steep stream side slopes is typical of horizontally bedded shale. In a humid climate, gentle sloping, softly rounded hills are the typical forms of horizontal bedded shale, while a gently rolling surface broken by circular sinkholes is characteristic of limestones. A series of straight or curving ridges points to tilted interbedded sedimentary rocks.

Massive, rounded, unbedded domes like hills, often strongly jointed and with variable summit elevation, suggest that the bedrock consists of granite. A series of tongue-like flows that may overlap and interbed, often with associated cinder cones, is the typical landform of lava flows and material ejected from volcanic vents. A nearly level rock surface, often cut by major streams that form deep valleys and with columnar jointing on valley walls, indicates basalt.

Fan-shaped landforms occurring at the base of steep slopes where streams discharge into an area with a more subdued relief indicate alluvial soil deposits. Most fan materials consist of boulders, gravel and sand with some silt that have been eroded from higher elevations and transported downslope.

An overall level relief, a gentle downstream gradient and meandering rivers with cut-off meanders and natural levees is the typical landform of flood plains. The texture of flood plain soils varies greatly, because they have been accumulated slowly over years of shifting stream courses and overbank flooding. Deposits from overbank flooding are usually poorly drained silts and clays. Cut-off meanders contain standing water or poorly drained organic soils. Deposits inside of river curves (point bar deposits) consist mainly of sand and gravel. Natural levees contain principally sand and silt.

A nearly level surface formed where streams discharge into a lake or into the ocean identifies a delta. Some deltas contain a great deal of sand and gravel. Other deltas have extremely variable soil conditions, as in the case of river flood plains.

Mounds, hills and ridges with a wind-swept appearance and an asymmetric cross-section are likely to be sand dunes. They are found inland from sandy beaches with onshore winds and in desert areas where the disintegration of sandstone provides the material.

Drainage

Rocks and soils have characteristic drainage conditions depending on surface runoff, permeability and internal drainage.

A dendritic drainage pattern (Figure 12.1) with gently curving streams is typical of horizontally bedded sandstone. Centripetal drainage into sinkholes and very few surface streams are found in terrain with horizontally bedded shale. A trellis drainage pattern (Figure 12.1) with major streams running along valley bottoms and secondary streams flowing down scarp slopes and joining the major streams at right angles, indicates tilted interbedded sedimentary rocks.

A coarse-textured dendritic drainage pattern, with a tendency for streams to curve around the base of dome-like hills and secondary channels following joints, is found where the bedrock consists of granite. A rudimentary drainage pattern is associated with lava flows. Also, very few surface streams are visible where the bedrock is basalt.

A limited surface drainage system, but numerous distributary (constructional) channels, is typical of alluvial fan deposits. A principal stream with few connecting secondary streams is associated with flood plains. A major channel with branches extending in a fan-shaped pattern or in random directions is seen in deltas. Complete absence of surface drainage is characteristic of sand dunes.

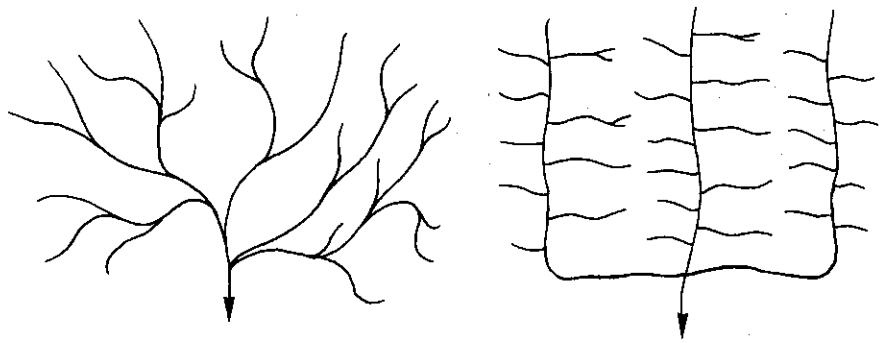


Figure 12.1 Dendritic and trellis drainage pattern.

Erosion

The smallest drainage features that can be seen on aerial photos are gullies. Gullies result from erosion of unconsolidated material and develop where rainfall cannot percolate adequately into the ground. Gullies collect in small rivulets and take on a shape characteristic of the material in which they are formed (Figure 12.2).

In sand and gravel, gullies tend to be short with a V-shaped cross-section. In silty soils gullies develop with a U-shaped cross-section, and in clay soils gullies are normally long with a rounded cross-section.

Photo tone

The absolute value of the photo tone (brightness) on an air photo depends on certain terrain characteristics but also on such factors as photographic exposure, sun angle and cloud shadows. Because of the effect of non-terrain related factors, interpretation of photo tones must rely on relative tone value rather than absolute values.

For bare soils a light tone is typical of coarse-textured soils with low moisture content and low content of organic matter. A dark tone is characteristic of fine-textured soils with high moisture content and of organic soils. Coarse-textured soils will generally have a sharper borderline between light and dark tones than fine-textured soils. As the photo tone is directly dependent on the moisture content in the surface of the soil, interpretation of photo tones is difficult in arid as well as in very humid regions.

Vegetation

Differences in natural or cultivated vegetation often indicate differences in terrain conditions, but vegetation may also obscure differences in terrain conditions. Therefore, interpretation of vegetation should be done with great caution.

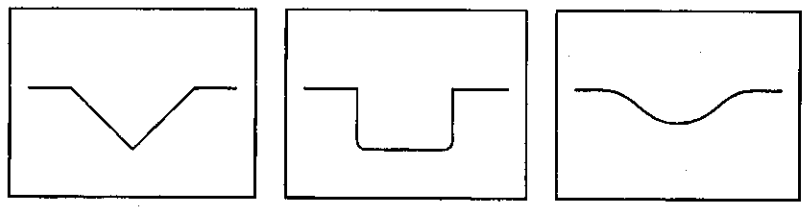


Figure 12.2 Typical gully cross-sections.

12.2.3 Satellite image interpretation

During the last two decades satellite image interpretation has come into use for geological studies and soil investigations. Satellite images are employed as a supplement to air photos or as a substitute for air photos in cases where air photos are not available. The images of most use for soil investigations are the images produced by the Landsat satellites.

Landsat

The first of the Landsat series of satellites was launched by NASA in 1972. Since then several hundred thousand images of the earth's surface have been transmitted to receiving stations operated in different countries. The sensors record information over very large areas, each scene covering 185 × 185 km. The first Landsat satellite had a resolution of 80 m. Later satellites have been able to produce images with a resolution of 30 m.

Most Landsat images are produced by scanning in four wavelength bands, two in the visible spectrum of green and red, and two in the reflected infrared. The satellite data are provided on films or on digital tapes. Films are normally produced at scales ranging from 1:1000000 to 1:250000. In addition to black and white images of single bands, 'false' colour composites are available. The composites are generated by registering three bands on to colour films. Generally, the green band is printed in blue, the red band in green, and one infrared band in red. This combination simulates the colour rendition of infrared colour films.

Photographic products are preferred by most users, but the inherent digital form makes satellite data amenable to computer analysis. Quantitative techniques can be applied to automatically interpret the digital image data. Prior to displaying image data for visual analysis, enhancement techniques may be applied to accentuate the apparent contrast between features in the picture.

SPOT

The French/Swedish SPOT satellite was launched in 1986. The SPOT satellite has the capability of collecting data similar to those from Landsat, with somewhat better resolution. Further, the SPOT satellite is able to collect stereoscopic panchromatic images.

Use of satellite imagery

Because of the large scale and small resolution, satellite images are most useful for the analysis of regional features, such as terrain types, major geological boundaries and catchment areas. As with air photos the interpretation of satellite images requires experience and interpretation should always be checked by field investigations.

Satellite images are normally obtained from existing archives, as it is rather expensive to have images specially commissioned. The bulk of existing satellite images is produced by the Landsat satellite, but the number of SPOT images is growing.

12.2.4 Terrain classification

Over the last 30 years, different Terrain Classification systems have been developed to provide a basis for natural resource surveys in the absence of adequate, detailed thematic maps. The use of Terrain Classification is based on the fact that the landform is an important factor in the soil formation. Each landform in a particular area has its own geology. When classifying a terrain, typical features of the terrain are identified by the patterns they make in aerial photos and satellite images.

These features are then used as mapping units. The main advantage of Terrain Classification is that the amount of soil sampling required to characterize an area is minimized. Terrain Classification is particularly helpful in tropical and sub-tropical regions where most soils are residual soils, i.e. they have been formed over or near the rocks from where they originate.

In many parts of the world Terrain Classifications already exist, often prepared for agricultural land use surveys (Figure 12.3). A system of Terrain Classification recommended for engineering use is described in a manual (ref. 2) published by the Transport Research Laboratory (TRL). This system is similar in concept to that used by other organizations, and land system maps made for other purposes can also be used for engineering surveys. Terrain maps are commonly produced at scales of 1:250 000–1:1 000 000.

Two levels of mapping units are normally used in the TRL Terrain Classification System: land systems, and land facets.

Land system

A land system is defined as a large area with a recurring pattern of landforms, soils and drainage. Its physical characteristics give it a distinctive, unified character, recognizable from the air.

Land facet

Land facets are the basic units that make up land systems. A land facet is a terrain unit of uniform slope, parent material, soil, and drainage pattern. The same road design and construction cost is supposed to be applicable to the whole extent of a land facet. Each land facet is subdivided into land elements often too small to be mapped.

The land systems and land facets are mapped on air photo mosaics of the project area. The principle in the classification method is illustrated in the block diagram in Figure 12.4.

12.3 FIELD INVESTIGATION

12.3.1 Geophysical exploration

Methods

Two geophysical methods of soil exploration are being used for highway purposes. These are the electrical resistivity and the seismic refraction methods.

Electrical resistivity

The electrical resistivity method is based on the fact that different soils may have different electrical resistivity. The resistivity mainly depends on content of clay minerals, moisture content, and type and concentration of electrolyte in the soil-water. An increasing content of clay, water or electrolyte causes decreasing resistivity.

In performing the test, four electrodes are inserted in the surface of the soil and arranged on line symmetrically about a point (Figure 12.5). A direct current is made to flow through the soil between the two outer electrodes, and the drop in potential is measured between the two inner electrodes. If the soil is homogeneous, the specific resistivity ρ may be calculated as:

$$\rho = \frac{\pi a}{4}\left(\frac{L^2}{a^2} - 1\right)\frac{V}{I}$$

where:

a = distance between potential electrodes;

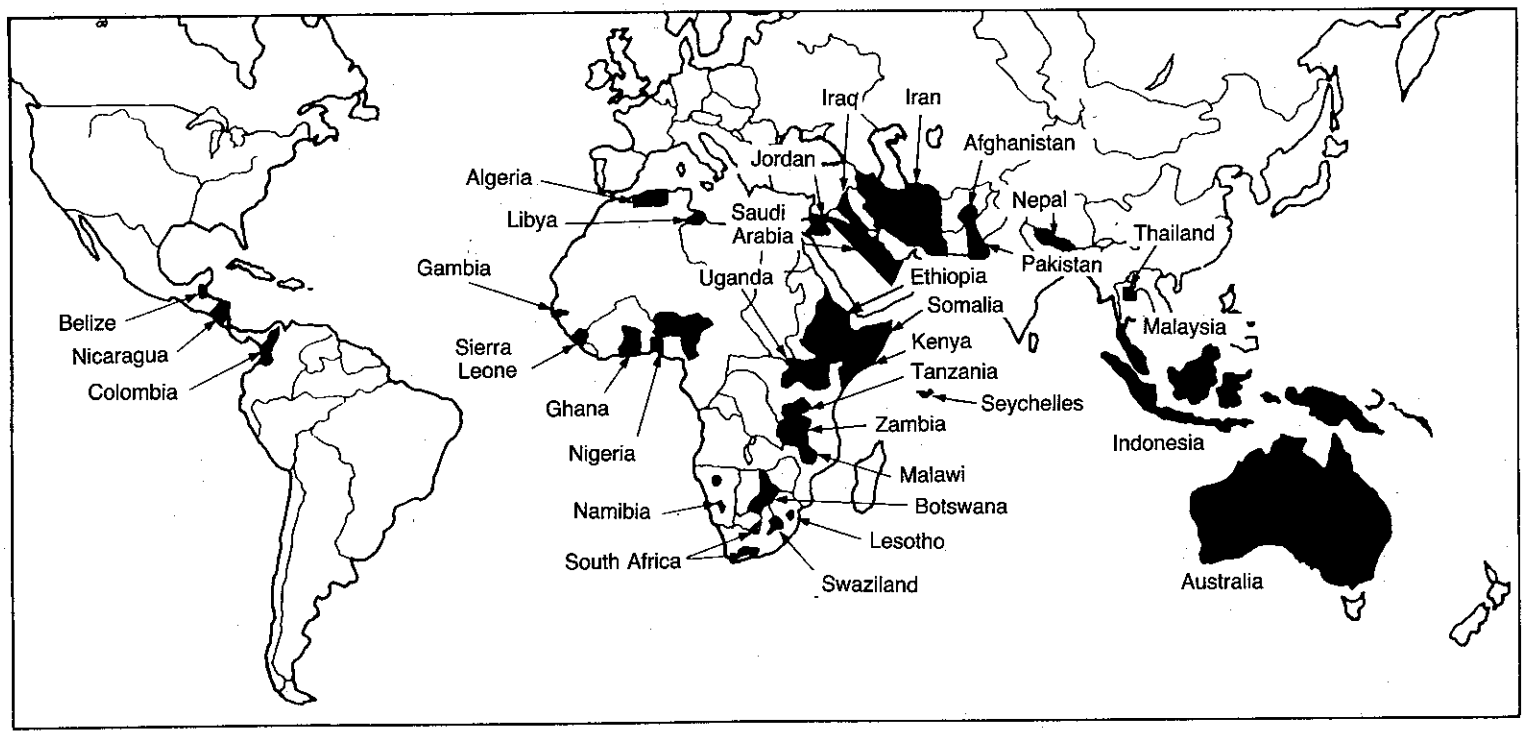


Figure 12.3 Areas covered by Terrain Classification (ref. 2). (Crown copyright; reproduced with the permission of the Controller of HMSO.)

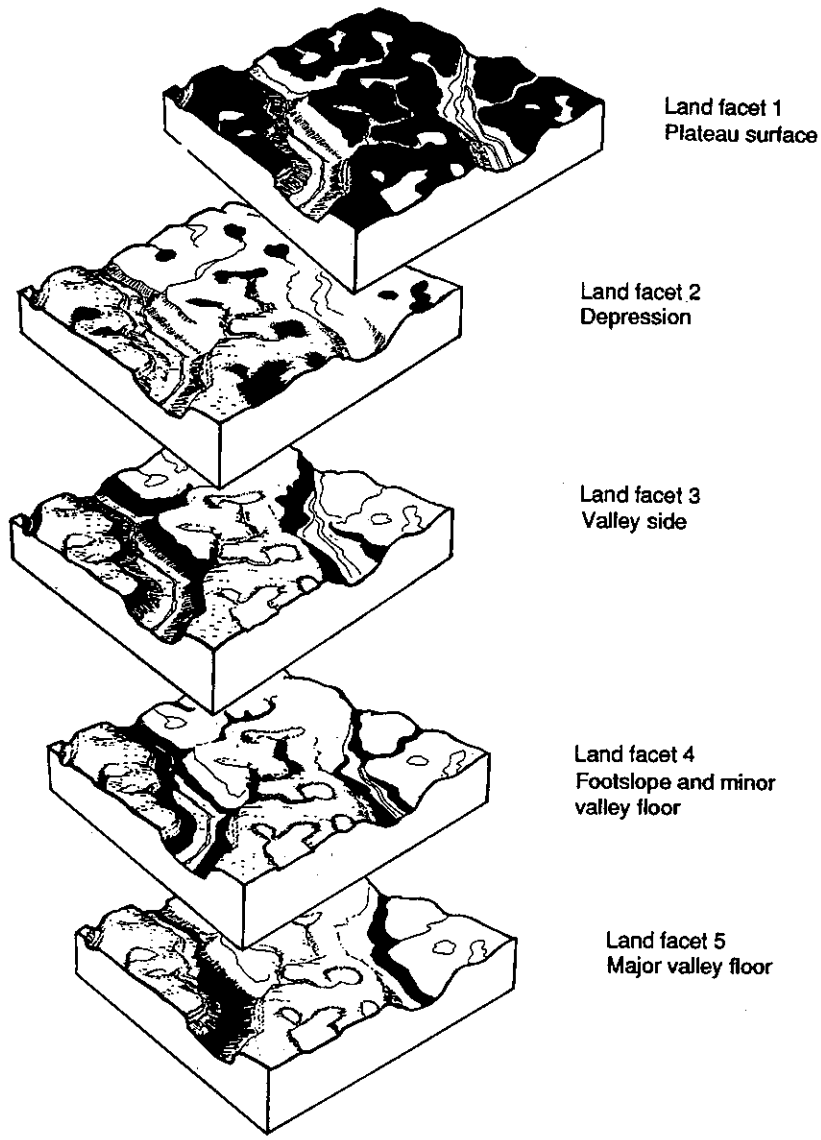


Figure 12.4 Block diagram of a land system with five land facets (ref. 2). (Crown copyright; reproduced with the permission of the Controller of HMSO.)

L = distance between current electrodes;
I = current intensity;
V = potential difference.

When the soil is heterogeneous, for example, made up of different layers, the calculated parameter is the apparent resistivity of the heterogeneous sequence down to a certain depth. This depth increases with increasing distance between the current electrodes.

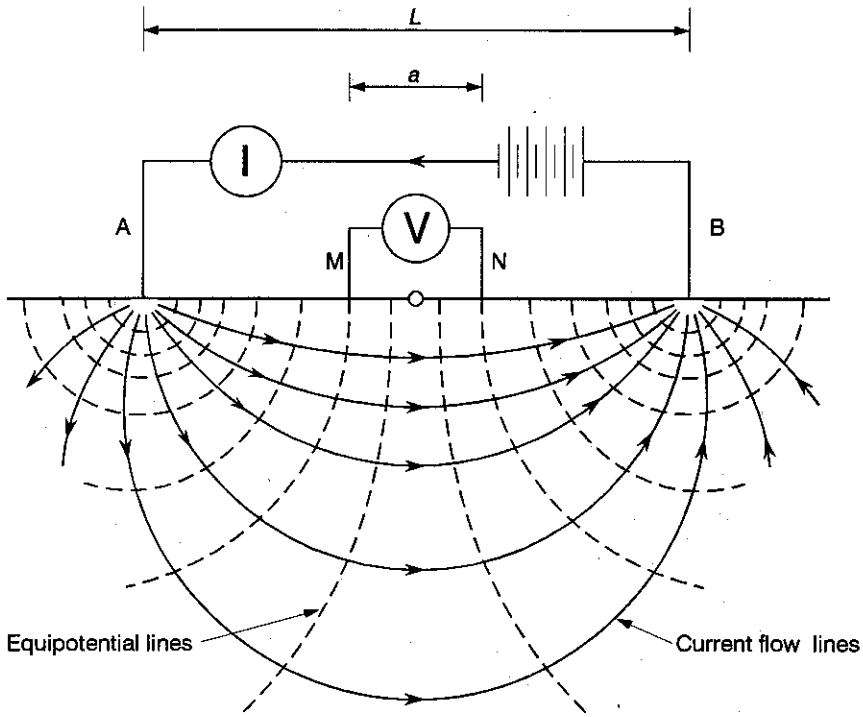


Figure 12.5 Equipotentials and current lines in a homogeneous earth.

The electrical resistivity method may be used for soundings or for line profiling.

Soundings

A sounding is carried out with an increasing electrode spacing. The centre of the configuration and its orientation remain fixed. The variation of the apparent resistivity with increasing electrode spacing reflects the variation of soil conditions. The variation in resistivity may be interpreted into a soil profile showing the supposed thickness and type of different soil layers in the range of measurements. Some previous knowledge of the occurring soil types is necessary in order to obtain a reliable interpretation. However, an electrical sounding has a much bigger range of measurement than an auger boring, and the costs of making electrical soundings are relatively small. A combination of electrical soundings and a few calibrating auger borings may be a very cost-effective way of soil exploration.

Line profiling

A line profile shows the horizontal variation of the apparent resistivity for a fixed electrode configuration moved along a line. All measurements along the line correspond to the same approximate depth of penetration of the electrical field. If an area is covered with a number of line profiles, it is possible to produce a resistivity map. On the resistivity map curves are drawn through points with the same resistivity. On a site with moist, clayey surface deposits a resistivity map may reflect the distance to stable soil strata. In sand and gravel deposits the resistivity map will reflect the thickness of the deposits unless the measurements are disturbed by a groundwater table. In order to translate a resistivity map into layer thicknesses and examine the quality of the materials it is necessary to make a few calibrating borings in selected locations.

Seismic refraction

The seismic refraction method relies on the principle that the velocity of sound in soils and rocks is different for different materials. A shock wave is created by detonation of a small explosive charge on the surface of the terrain. The time intervals required for the shock wave to reach several detectors placed in line at different distances from the source are recorded. If the soil is uniform for some depth, these time intervals are directly proportional to the distance from the point of detonation. If a substratum in the earth's crust has a higher velocity of sound than the overburden, then the time interval to more distant points is shortened. The shock wave will travel down to the substratum, through this and then up again through the overburden. By plotting the time of travel against distance from the point of detonation the depth to the substratum can be calculated. The seismic refraction method is particularly useful in predicting the depth to bedrock.

12.3.2 Sampling

Methods

Soil samples near the surface are normally taken by use of hand-tools, such as shovels and hand-operated augers. At greater depths than two metres or so it is necessary to use power augers with appropriate samplers. Undisturbed soil samples are obtained by use of a core cutting device. Rocks from prospective quarries are sampled by selection of representative boulders.

Wash drilling

In developing countries wash drilling is often employed for soil investigations. Wash drilling is relatively cheap and primarily used for well drilling. The equipment uses pressurized water to wash the drill rod down. Great precaution should be taken when using wash drilling for soil sampling. The water pressure causes extensive stirring of the soil in the vicinity of the bore hole, and it may be difficult to extract undisturbed samples from the bore hole even after careful cleaning of the hole.

Location

Samples of soils and rocks are extremely small compared to the quantities they represent. For example when sampling subgrade soils, one 10 litre sample may be taken for every 100 m^3 of soil. That is only 1/10 000 of the volume. Therefore, it is of paramount importance that all samples are taken as representatively as possible. As explained earlier the sampling should be planned using topographic maps, geologic maps, air photos, etc. Geophysical investigations are also a useful guide to selecting suitable locations of test pits and borings.

Depth

The depth of test pits and borings for road-beds should be at least 1 metre below the proposed subgrade elevation. Where soft soils are encountered, it may be necessary to increase the depth down to a denser stratum. Borings for structures or embankments should extend below the level of significant influence of the proposed load. Where drainage may be influenced by either pervious, water-bearing materials or impervious materials that can block internal drainage, borings should be sufficient in number and depth to outline the required quantities of material.

Number of samples

In order to secure representative samples it is advisable to take a greater number of samples in the field than can possibly be tested in the laboratory. The field samples are then classified visually, and typical samples are selected for testing.

12.4 LABORATORY TESTING

Sample reduction

The size and type of samples required are dependent upon the test to be performed. Normally, it is necessary to reduce the field samples into one or more smaller laboratory samples. Reduction of disturbed samples should always be carried out by quartering (Figure 12.6) or by use of a sample splitter.

Test methods

In specifications for testing of soils and aggregates reference is frequently made to British Standards (BS) (ref. 3), standards issued by the American Society for Testing and Materials (ASTM) (ref. 4), and standards specific to highway engineering issued by the American Association of State Highway and Transportation Officials (AASHTO) (ref. 5). To a large extent the same type of methods are used all over the world. However, when actually performing tests it is of the utmost importance that the specified standards be followed precisely, as small differences in the testing procedure may have a noticeable influence on the test result.

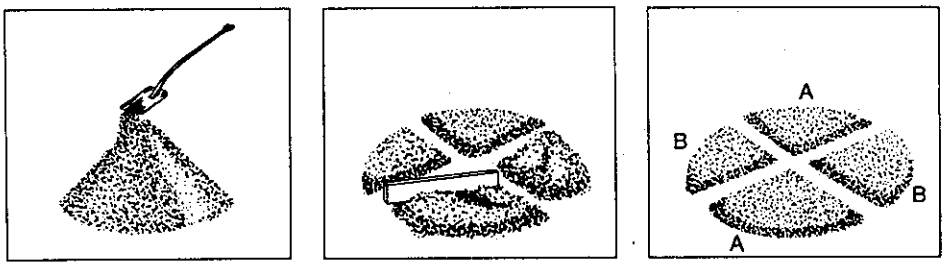


Figure 12.6 Quartering.

The purpose and the general principle of the most common tests are briefly described in the following sections.

12.4.1 Testing of disturbed soil

Particle size distribution

The distribution of particle sizes in soils is important in road engineering as the value of many properties, such as internal friction, voids content, wear resistance and permeability, depend on the gradation. The distribution of particle sizes larger than 75 μm is determined by sieving a sample through a number of standard sieves.

According to most test standards the soil sample should be placed on a 75 μm sieve and washed free of all fine material, before the sample is dried and sieved. Alternatively, the soil should be washed through successive sieves with water. However, many laboratories practice dry sieving of unwashed samples. In some tropical soils the fines tend to stick to the coarse particles. Therefore, dry sieving should only be allowed if it has been shown that the same results are obtained as with wet sieving or sieving of washed samples.

The sieves may be shaken by hand, but a mechanical shaker ensures more reliable results. All sieves are worn by use. Particularly the small sieve sizes are vulnerable, and they should be inspected regularly.

The results of a sieve analysis are normally visualized in a graph. The sieves are plotted on a logarithmic scale as the abscissa. The mass proportions of the soil

sample passing the corresponding sieves are plotted on an arithmetic scale as the ordinate. Some typical particle size distribution curves or sieve curves are shown in Figure 12.7.

A well-graded soil is one with a gently sloping sieve curve indicating that the soil contains a wide range of particle sizes. A uniformly graded soil is one with a predominance of single-sized particles. A gap-graded soil has one size range of particles missing.

For purposes of succinct communication the coefficient of uniformity is sometimes used as a single numerical expression of particle-size distribution. The coefficient of uniformity is defined as the ratio of the sieve size through which 60% of the material passes, to the sieve size through which 10% passes.

Water content

The engineering properties of a soil, e.g. strength and deformation characteristics, depend to a very large degree on the amount of voids and water in the soil, i.e. the condition of the soil. The water content is defined as the weight of water contained in a soil sample compared with the oven-dry weight of the sample. It is customarily expressed as a percentage, although the decimal fraction is used in most computations.

Specific gravity

The specific gravity of a soil is used in the equations expressing the phase relation of air, water and solids in a given volume of material. The specific gravity of a soil is calculated as the ratio between mass and volume of the solid particles of a sample. The volume of the particles is determined by placing the sample in a volumetric bottle (pycnometer) filled with water and measuring the volume of displaced water.

Plasticity limits

The plasticity limits are used to estimate the engineering behaviour of clayey soils and form an integral part of several engineering classification systems. The plasticity limits include the liquid limit and the plastic limit, and they are determined by arbitrary tests on the fine soil fraction passing the 0.42 mm sieve.

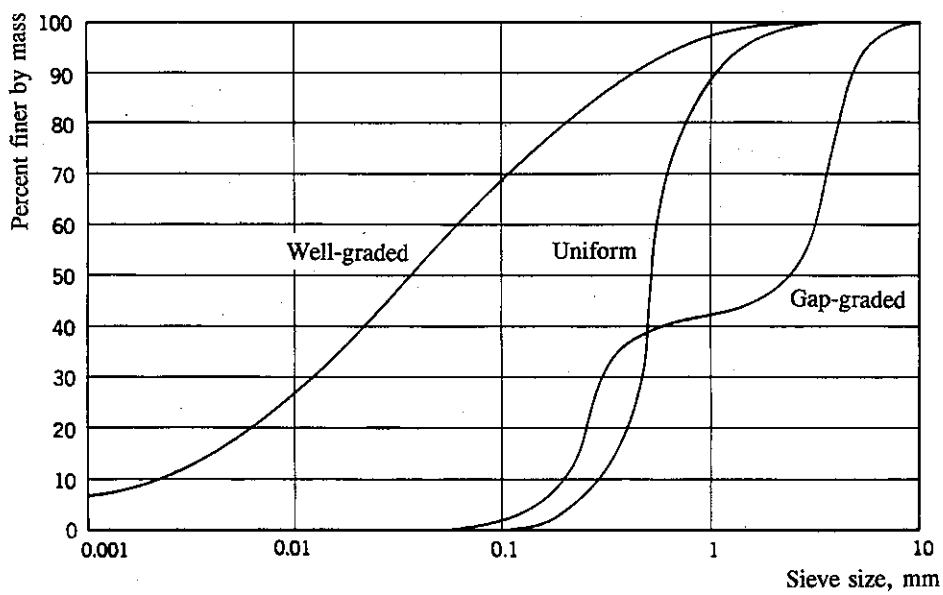


Figure 12.7 Particle size distribution curves.

Liquid limit

The liquid limit (LL) is determined by performing trials in which a sample is spread in a metal cup and divided in two by a grooving tool. The liquid limit is defined as the water content of the soil that allows the divided sample to flow together, when the cup is dropped 25 times on a hard rubber base.

Plastic limit

The plastic limit (PL) is determined by alternately pressing together and rolling a small portion of soil into a thin thread causing reduction of the water content. The plastic limit is defined as the water content of the soil when the thread crumbles.

Plasticity index

The difference between the liquid limit and the plastic limit is called the plasticity index (PI).

Plasticity product

In some countries, the plasticity index and the percentage of fines are combined into a plasticity product. This approach has considerable merit, as the effective contribution of the plasticity of the fines to the performance of the whole material depends on the proportion of fines.

Test specimens for plasticity testing are often prepared by mixing oven-dried material with water. However, the plasticity of certain tropical soils decreases, when the soil is dried before testing. Therefore, it may be necessary to prevent drying of the material and use a 'wet preparation' of the test specimens. Other soils may contain weak aggregations that break down under intense mixing with an attendant increase in plasticity. This also calls for wet preparation, as this procedure involves less mixing than dry preparation.

Free swell

Expansive clays are a problem in many regions in the tropics. A simple test which can be used to verify swelling tendencies is described in ref. (6). A measured volume of dry, pulverized soil is poured into a graduated glass containing water. After the soil comes to rest at the bottom of the cylinder, the expanded volume is measured. The free swell is calculated as the increase of volume as a percentage of the initial volume.

Field density

The field density (in-place density) has a great influence on the bearing capacity and the settlement of soils. For that reason soil compaction is an important component of road construction. Measurements of field density are made during soil investigation, but most measurements are taken for the purpose of compaction control during construction. Several methods are used for determining field density.

Drive cylinder

The most simple method is the drive cylinder or core cutter method where a fixed volume of soil is removed by driving a thin-walled cylinder down into the soil (Figure 12.8). The sample is brought to the laboratory, and the dry weight determined. The dry density is calculated by dividing the oven-dried mass of the

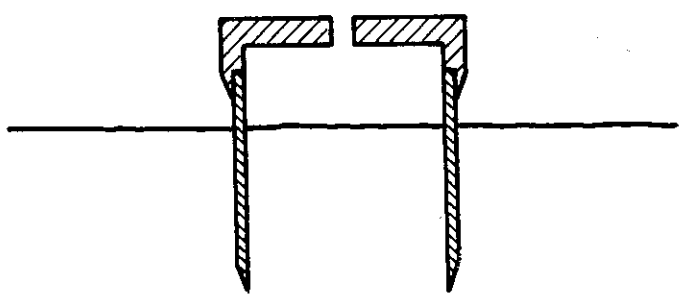


Figure 12.8 Drive cylinder.

soil specimen by its volume. The method is not applicable to friable soils and soils containing coarse material.

Sand cone

The sand cone or sand replacement method is widely used to determine the density of compacted soils. A sample is removed by hand excavating a hole in the soil. The *in situ* volume of the sample is determined by measuring the volume of dry, free-flowing sand necessary to fill the hole. A special cone is used to pour the sand into the hole (Figure 12.9). The dry weight of the sample is determined in the laboratory. The method is not recommended for soils that are soft, friable or in a saturated condition. The method is rather time-consuming.

Rubber balloon

In the rubber balloon method the volume of the test hole is determined by measuring the volume of water necessary to fill the hole after a thin, elastic, watertight membrane (balloon) has been inserted in the hole. A slight external pressure is applied to the water to ensure complete filling of the test hole.

Nuclear density gauge

The nuclear density gauge utilizes gamma rays to measure wet density (total density) and neutrons to measure water content. The results are available on the spot in a matter of minutes after the test has been completed. Strictly speaking, the method is not a laboratory test. It is necessary to calibrate the results with direct

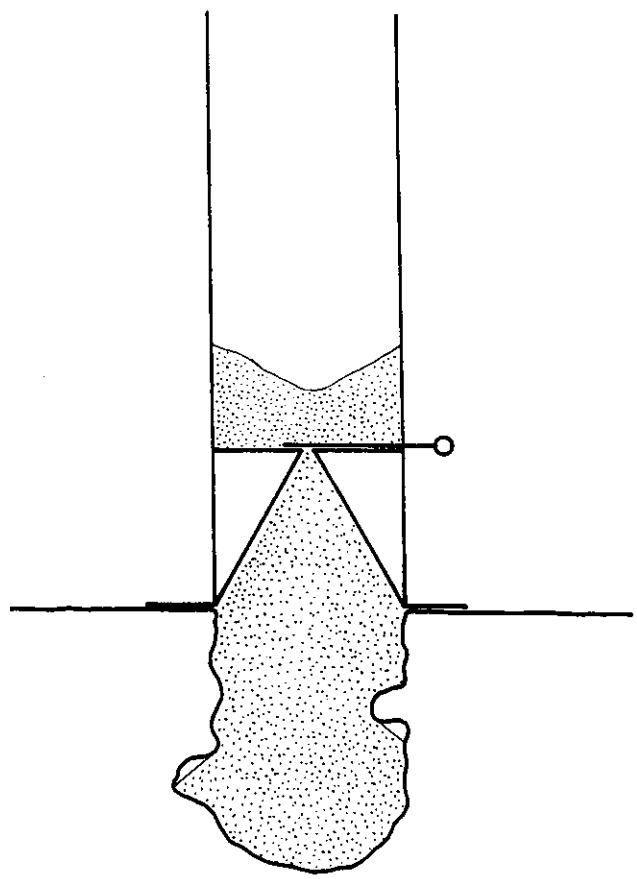


Figure 12.9 Sand cone.

measurements of density and water content of each soil type encountered on the site. The nuclear density gauge is potentially hazardous, and care is required in the handling of the equipment. It is costly and mainly used for compaction control on big, equipment-based road work.

Laboratory compaction

The level of compaction to be achieved in the field during construction is normally specified as a percentage of the maximum dry density obtained in a compaction test in the laboratory. The traditional laboratory tests are the standard and the modified AASHTO compaction. In British Standards they are named light and heavy compaction. From the man who invented the laboratory compaction tests they are also known as standard and modified 'Proctor tests'.

A sample of the soil is compacted in a cylindrical metal mould having an approximate volume of 1 or 2 litres (diameter 10 or 15 cm). Compaction is achieved by use of a falling hammer. For modified compaction a heavier hammer plus more and longer hammer drops are used than for standard compaction. The compactive effort is supposed to be equivalent to a medium-sized roller in the field. Modified compaction better represents what can successfully be obtained in the field, but standard values are widely used, particularly for subgrades.

The compaction test is repeated for a number of different values of water content in the soil. For each test the achieved dry density is measured. Corresponding values of water content and dry density are plotted in a diagram, and the points connected with a smooth curve (Figure 12.10).

Normally, the compaction curve has an obvious peak, i.e. there is an optimum water content at which the maximum dry density will be achieved for a particular compaction effort. For the same soil higher compaction effort results in a higher maximum dry density at a lower optimum water content. For the same compaction effort gravelly soils have a higher maximum dry density and a smaller optimum water content than clayey soils.

Certain tropical soils contain weak aggregations that break down under intense

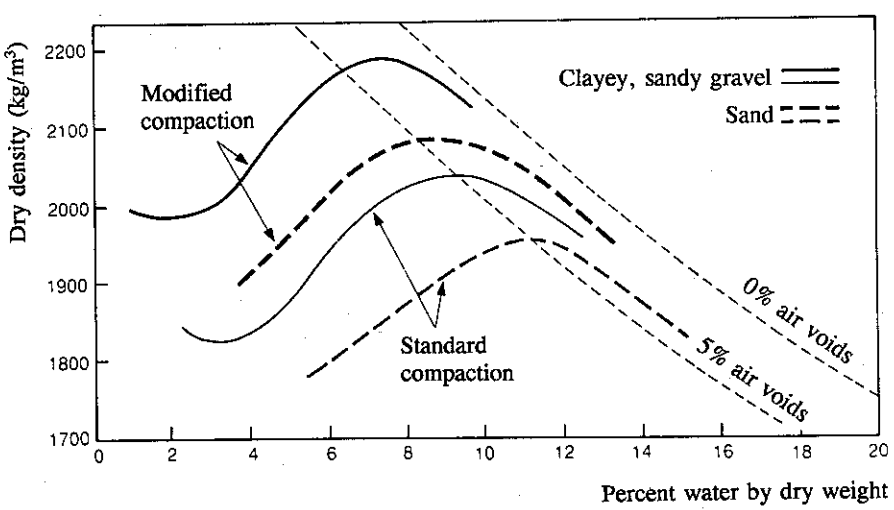


Figure 12.10 Example of compaction curves (ref. 7). (Crown copyright; reproduced with the permission of the Controller of HMSO.)

remoulding. For these types of soils fresh material should be used for each point on the compaction curve. As an alternative the laboratory testing may be replaced by full-scale testing in the field. The tendency to degradation may be estimated by a sieve analysis before and after a compaction test.

Because of the limited dimensions of the test equipment, the AASHTO compaction tests are not suitable for soils containing substantial quantities of coarse particles. When using the 1 litre mould, material retained on the 4 mm sieve is normally discarded. When the 2 litre mould is used, the maximum particle size is increased to 16 or 20 mm. Compaction tests on soils containing many large particles are discussed later in the section dealing with testing of aggregates.

The optimum water content is not a constant for a particular material. The optimum water content is dependent on compaction effort and compaction method as well as on soil type. During construction of major highways, compaction in the field is achieved by rolling. In order to obtain a high dry density it is common practice to try to bring the water in the soil close to the optimum corresponding to the applied compaction effort and method. If a soil layer is too wet, it is left exposed to wind and sun for some time and allowed to dry. If the soil layer is too dry, the moisture content is increased by spraying with water.

In dry tropical regions it may be difficult to obtain the quantity of water necessary to increase the water content in a soil layer to the optimum. Under those circumstances it may be favourable to compact the soils in the dry state (dry compaction). For fine grained soils a small water content creates cohesion between the particles that impedes the compaction. This is illustrated as a minimum in the beginning of the compaction curves in Figure 12.10.

California bearing ratio

The California bearing ratio (CBR) test is the most common test for evaluating the bearing capacity of subgrade soils. It measures the force needed to cause a plunger to penetrate 2.5 or 5 mm into a soil sample compacted into a 2 litre cylindrical mould (diameter 15 cm) as shown in Figure 12.11. The measured force is taken as a percentage of a standard force.

One of the main problems with CBR testing is deciding what water content to use in the soil sample. In many countries the standard procedure is to compact the soil at optimum water content and then soak the sample in water for four days. However, four days of soaking would be very conservative in arid and semi-arid areas. On the other hand, in wet areas four days of soaking may not be enough with clayey soils of low permeability. Normally, it is more useful to derive the CBR from tests on samples where the water content has been adjusted to the critical level likely to occur in the field. Selection of proper water content is discussed in Chapter 16.

Equipment is available to carry out *in situ* CBR testing in the field. The equipment is attached to a truck, and the truck used as a dolly. For practical reasons *in situ* equipment is normally only used on existing roads where the base, subbase, and subgrade are exposed by digging of holes.

The CBR test is of poor reproducibility, particularly with granular soils.

Dynamic cone penetration

The dynamic cone is a quick and cheap alternative to *in situ* CBR tests. The cone consists of a steel core with a 30° point angle. The cone is forced into the soil by use of a drop hammer, and the penetration (DCP) is measured in mm per blow. Empirical relations between penetration and CBR may be derived, but the DCP is even further removed from reality than the CBR.

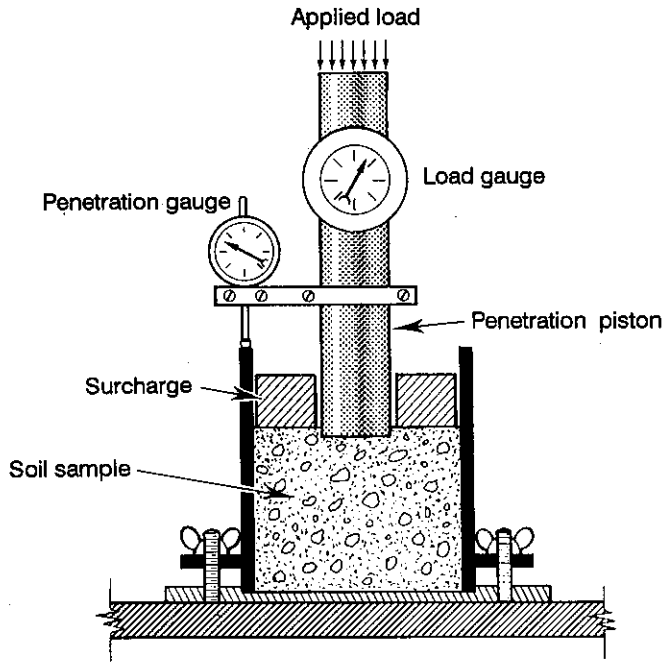


Figure 12.11 Equipment for CBR testing.

12.4.2 Testing of undisturbed soil

Undisturbed samples are normally used for consolidation testing and triaxial compression testing. The consolidation test is employed to estimate the settlement of soils under the load of an embankment or other structure. The triaxial compression test is used to examine the structural strength of soils as foundations for structures or in detailed studies of slope stability problems. Consolidation testing and triaxial testing in particular require complicated laboratory equipment and expertise.

12.4.3 Testing of aggregates

Particle size analysis

Particle size analysis on aggregates is carried out using the same procedure as described for soils. Circular sieves with a frame diameter of 200 mm are normally used for analysis of soils and fine aggregate. However, for analysis of coarse aggregate it is useful to employ sieves with a frame diameter of 300 mm or more, because bigger samples are needed in order to obtain representative results.

An important use of the sieve curve is for estimating the volume occupied by different fractions of the soil. In some types of natural gravel, particularly laterite, there may be a significant difference between the specific gravity of the coarse and the fine particles. For this type of soil it may be useful to convert mass proportions to volume proportions when plotting the sieve curve.

Specific gravity

The specific gravity of aggregates is used for converting mass to volume.

Volume calculations of aggregates are primarily used in connection with mix design for cement- and asphalt concrete. The test procedure is similar to that described for soils except that bigger samples and bigger pycnometers are needed for coarse aggregate. Instead of using a pycnometer the volume of the sample may be determined by placing the sample in a wire basket and weighing it before and after immersing in water.

Water absorption

High porosity of aggregates may be a sign of low mechanical strength. Furthermore, aggregates with high porosity may be difficult and costly to dry during processing of an asphalt hot mix. The porosity is estimated by measuring the water absorption.

The water absorption is determined by immersing a dry sample in water for 24 hours. The surfaces of the particles are then dried by rolling the sample gently in a dry cloth. The water absorption is calculated as the difference in weight between the saturated, surface-dry sample and the dry sample as a percentage of the weight of the dry sample.

Sand equivalent

The sand equivalent test is useful for evaluating the plastic properties of the sand fraction of aggregates. A volume of damp aggregate passing the 4.75 mm sieve is measured. The material should not be dried before testing as this may change the properties. The sample and a quantity of flocculating solution (calcium chloride solution) are poured into a graduated glass and agitated. After a prescribed sedimentation period, the height of sand and the height of flocculated clay are read. The sand equivalent (SE) is the height of sand as a percentage of the total height of sand and flocculated clay.

Field density

Field density tests are used in construction to evaluate the compaction achieved in aggregate base and subbase. Common methods are sand cone and nuclear density gauge as described in the section dealing with testing of soils. However, measurements of field density are not very precise when dealing with coarse material. In some countries this has led to the use of method specifications for compaction instead of end product specifications, see Chapter 14.

Vibro compaction

AASHTO compaction tests are not suitable for testing materials with a high content of coarse particles. For coarse materials vibro compaction is a more useful method. A sample of the aggregate is placed in a big cylindrical metal mould having an approximate volume of 14 litre (diameter 28 cm). Compaction is achieved by applying a surcharge to the surface of the soil, and then vertically vibrating the mould, soil and surcharge. The equipment is heavy and requires a laboratory provided with lifting devices. Only a few soils laboratories are able to perform vibro compaction. The best alternative is to make full-scale compaction tests in the field using appropriate rollers.

California bearing ratio

The CBR test is unsuitable for testing of crushed stone and coarse gravel, because of the need for removing particles bigger than 20 mm. For design purposes the CBR is sometimes estimated based not on testing but on previous experience combined with evaluation of the particle size distribution curve and the particle shape.

Particle shape

The particle shape influences the compaction and strength characteristics of aggregate mixtures. Cubic particles are less workable but more stable than flaky and elongated particles. The particle shape test is performed on coarse particles such as particles retained on the 6 mm sieve. Each particle is gauged by use of a length gauge. Particles having their smallest dimensions less than 0.6 of their mean size are classified as flaky. Particles having their largest dimension more than 1.8

times their mean size are classified as elongated. The mean size is defined as the mean of the two sieve sizes between which the particle is retained in a sieve analysis. The percentages by mass of flaky particles in a sample is called the Flakiness index.

Soundness

The soundness test is used in survey and design to estimate the soundness of aggregate when subjected to weathering action. This is accomplished by repeated immersion in saturated solutions of sodium or magnesium sulphate followed by drying. The internal expansive force, derived from the rehydration of the salt upon re-immersion, simulates the weathering action. The sample is sieved before and after the simulated weathering action, and the percentage of loss for each fraction is calculated. The precision of the test is poor, and it may not be suitable for outright rejection of aggregates without confirmation from other tests.

Sulphate content

Sulphate in aggregates for Portland cement concrete is absolutely destructive. Materials containing sulphate may occur in deserts and semi-deserts. Aggregates are tested for sulphate by washing a small sample with hydrochloric acid. The hydrochloric acid converts the sulphate to sulphuric acid. Then the sample is filtered, and a barium chloride solution is added to the filtrate. The barium chloride reacts with the sulphuric acid forming insoluble barium sulphate. The sulphate content is calculated from the mass of dry precipitate.

Los Angeles abrasion

The Los Angeles abrasion test is intended to give an indication of the abrasion resistance in combination with the impact strength of coarse aggregates. The test is used for selecting the most suitable aggregate sources for quarrying. A sample, together with a number of steel balls, is loaded into a steel drum revolving on a horizontal axis as shown in Figure 12.12. The Los Angeles abrasion value is the

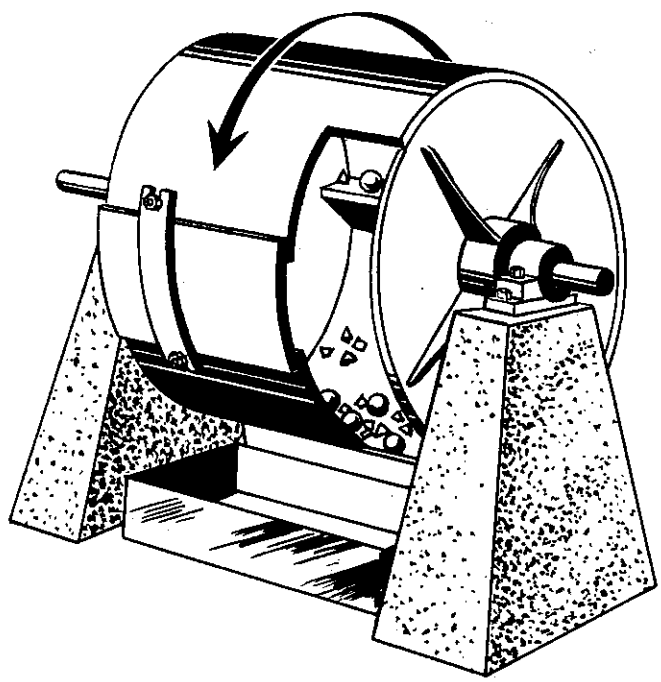


Figure 12.12 Los Angeles abrasion machine (ref. 8).

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse-Grained Soils More than 50 % retained on No. 200 sieve	Gravels More than 50 % of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5 % fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3^E$	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3^E$	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12 % fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50 % or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5 % fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3^E$	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3^E$	SP	Poorly graded sand[I]
		Sands with Fines More than 12 % fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50 % or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay[K,L,M,N] Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic silt[K,L,M]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay[K,L,M,P] Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12 % fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

[D] Sands with 5 to 12 % fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15 % sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15 % gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30 % plus No. 200, predominantly sand, add "sandy" to group name.

[M] If soil contains ≥ 30 % plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.

Figure 12.13 Unified Soil Classification System (ref. 4)

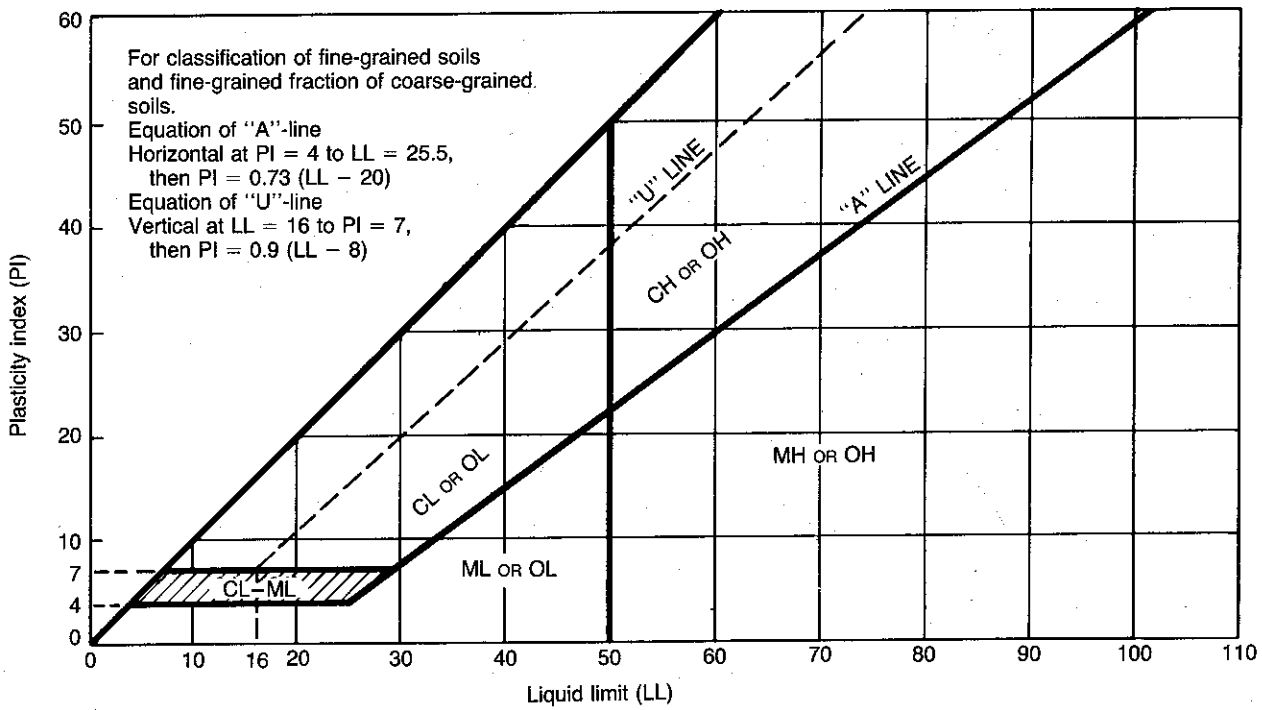


Figure 12.13 *Continued*

General Classification	Granular Materials (35% or less Passing 0.075 mm)							Silt-Clay Materials (More than 35% Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max.	—	—	—	—	—	—	—	—	—	—
0.425 mm (No. 40)	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
0.075 mm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.[a]
Usual types of significant constitutent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General Ratings as Subgrade	Excellent to Good							Fair to poor			

[a] Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30

Figure 12.14 AASHTO Soil Classification System (ref. 5).

percentage of fines passing the 1.7 mm sieve after a specified number of revolutions of the drum.

Aggregate impact

The aggregate impact test is a cheap alternative to the Los Angeles abrasion test. The sample is placed in a cylindrical steel mould and exposed to 15 blows from a falling hammer. The percentages by weight of produced fines passing the 2.36 mm sieve is called the Aggregate Impact Value (AIV). The AIV is normally about 105% of the so-called Aggregate Crushing Value (ACV) which is obtained by a similar test using a compressing machine instead of a falling hammer.

Polished stone

The resistance to polishing the Polished Stone Value (PSV), is an important factor when selecting suitable sources for aggregates to be used in bituminous surfacing. However, polishing tests are relatively complicated and only undertaken in special laboratories.

Organic impurities

The test for organic impurities is intended for approximate determination of the presence of injurious organic compounds in natural sands to be used in cement concrete. A quantity of sand and a volume of NaOH solution are added to a glass bottle and allowed to stand for a prescribed period. If the liquid turns darker than a standard colour, it indicates that the sand may contain deleterious organic matter.

Affinity for bitumen

Different test methods exist for determining the resistance to stripping, i.e. separation of a bitumen film from the aggregate through the action of water. However, none of the methods are very reliable. In the most simple method the resistance to stripping is evaluated by immersing an uncompacted sample of bitumen-coated aggregate in water. At the end of a soaking period the percentage of surface area of the aggregate on which the bitumen film is retained is estimated visually.

In the immersion-compression test the resistance to stripping is measured indirectly. Duplicate sets of test cylinders are compacted from bitumen-coated aggregate. The compression strength of the cylinders is measured, one set dry and the other after immersion in water. The difference in strength serves as a measure of the effect of moisture.

12.5 SOIL CLASSIFICATION

Soil classification

Soil classification is a way of systematically categorizing soils according to their probable engineering characteristics. The classification of a soil is based on its particle-size distribution and – if the soil is fine-grained – on its plasticity (LL and PI). The most widely used classification systems in highway engineering are the Unified Soil Classification System (Figure 12.13) and the AASHTO Classification System (Figure 12.14).

Soil classification should only be regarded as a means of obtaining a general idea of soil behaviour and it should never be used as a substitute for detailed investigations of soil properties.

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13 Tropical rocks and soils

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13.1 INTRODUCTION

Definitions

To the geologist the term 'rock' covers units of the earth's crust formed by certain geological processes. The term 'soil' is used to describe materials produced by disintegration of rocks. Many civil engineers restrict the term 'rock' to materials that cannot be excavated without blasting. The term 'soil' is then used more widely to describe all other naturally occurring materials. The engineering definition may cause confusion since modern equipment has made it possible to excavate many soft rock types without using explosives. In this chapter the geological terminology is applied.

Most roads are constructed on soils or soft rocks. Cuttings are made through soils occurring in the road line. Embankments are constructed from soils taken from cuttings or nearby excavations. The road pavement is placed on the local soil or on fill materials brought in. In mountainous regions hard rock may occur in cuttings, and rock fragments may be used as embankment fill. The pavement structure is mainly constructed from coarse-grained soils (sand and gravel). Hard rocks are usually quarried, crushed and graded to make aggregate for construction of heavy-duty road pavements.

The stability of cuts and fills is dependent on the properties of the soils and rocks concerned. The structural design of the pavement is dependent on the bearing capacity of the subgrade and the strength of the paving materials. It is obvious that thorough knowledge of the properties of the rocks and the soils in and near the road line is the basis for realistic road design.

Some rock types, such as reef limestone, are more widespread in the tropics than in temperate regions. Deeply weathered rocks are common in wet tropical areas. The soil-forming processes are more active and often continuous in the tropics and the technical properties of tropical soils may be quite different from the properties that characterize soils from regions with a temperate climate. This chapter briefly describes the principal rock types, the soil-forming processes, and the characteristic technical properties of the most widespread tropical soils. The aim is not to give a complete description of the rocks and soils in the tropics but to

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highlight some of the geotechnical problems that the road engineer working in these regions may be confronted with.

13.2 ROCKS

Rocks are divided into three groups: igneous, sedimentary and metamorphic (ref. 1). This classification indicates the means by which the rocks were formed. Igneous rocks are formed by cooling and solidification of hot molten rock material (magma). Sedimentary rocks are formed by consolidation and cementation of sediments that have been accumulated in water or deposited by wind. Metamorphic rocks are formed by the modification of igneous or sedimentary rocks as a result of pressure, heat and, occasionally, also as a result of chemical action.

13.2.1 Igneous rocks

Igneous rocks may be divided into two groups: extrusive and intrusive.

Extrusive rocks

Extrusive, igneous rocks are formed by rapid cooling of magma pouring out on the surface of the earth. The rapid cooling produces a fine-grained, often glass-like structure. Loose particles ejected from volcanic vents are called pyroclastic materials.

Intrusive rocks

Intrusive, igneous rocks are formed when magma solidifies within the earth's crust. Cooling of great volumes at great depths takes place very slowly and results in rocks with coarse-grained, crystalline textures. Cooling of smaller masses of magma in cavities and cracks near the earth's surface takes place more rapidly and produces medium-grained rocks or porphyries where both large and small crystals are present. Intrusive rocks are often exposed as a result of erosion of overlying materials.

Classification

Igneous rocks are commonly classified according to silica content and texture (Table 13.1). Normally, coarse- and medium-grained acid rocks are lighter coloured than their basic equivalents. Acid extrusive rocks, however, may be glassy and dark-coloured.

Table 13.1 Classification of igneous rocks.

		Type		
			Intrusive	
Acidity	*Amount SiO_2 (%)*	*Extrusive (fine-grained)*	*(medium-grained)*	*(coarse-grained)*
Acid	>65	Rhyolite	Porphyries	Granite
Intermediate	55–65	Andesite	Porphyries	Diorite
Basic	<55	Basalt	Dolerite	Gabbro
	<65	Trachyte	Felsite	Syenite

Engineering properties

In general, igneous rocks make good road aggregates. Fine-grained types have better abrasion and impact values but poorer polished stone values than coarse-

grained types of the same composition. Generally, the basic types have better affinity for bitumen than acid types. The common sources of road aggregate among igneous rocks are varieties of dolerite, basalt and granite.

Pyroclastics

Pyroclastic materials include dust, ash and cinders formed from lava blown apart by expanding gases as the magma neared the surface. The dust and fine ash may be carried for great distances by the wind before the materials settle on land or in water. The coarse cinders travel a shorter distance. Consolidated dust and ash is called tuff, while consolidated cinders are termed agglomerate.

Pyroclastic materials are normally porous and therefore have a low strength. These materials are not satisfactory for aggregate in concrete and asphalt, but experience from volcanic islands indicates that various types of cinders and agglomerates make suitable base course materials for roads.

Weathering

Rocks are disintegrated by weathering. Weathering includes physical disintegration caused by wind, water and freezing, and chemical disintegration caused by oxidation, reduction, dissolving and leaching. Expansion and contraction because of variation of temperature were formerly considered an important agent of weathering, but it has now been proved that this process requires water to work (ref. 1).

The weathering processes are much more severe in the tropics than in regions with a temperate climate. High temperatures and – in wet tropical areas – high humidity make the processes more active and often continuous. Consequently, it is commonly found that rocks exposed on the surface in wet tropical areas are deeply weathered. The deterioration may not be obvious in a hand sample, but the consequences of using such weathered rock in a road pavement may be disastrous (ref. 2). Therefore, igneous rocks in wet tropical regions should be subject to mineralogical investigation before a quarry site is opened, unless the quality of the rock is known from previous experience. It is not advisable to rely on the simple soundness test. The soundness test can be used to indicate whether rocks are weak and porous, but the test is less certain to identify rocks that are only partially weathered.

13.2.2 Sedimentary rocks

Sedimentary rocks are by far the most common rock type exposed at the earth's surface. The principal sedimentary rocks are sandstone, shale and limestone. Calcrete and silcrete also deserve mention.

Sandstone and shale

Sandstone and shale are rocks containing discrete particles derived from the erosion, transportation, and deposition of preexisting rocks and soils. Sandstone primarily contains sand-sized particles, while shales are composed of clay-sized particles.

Limestone

Limestone is formed in water from the sedimentation of shells and shell fragments or chemically from the precipitation of calcium carbonate. Limestone that contains a significant amount of magnesium carbonate is called dolomite. Chalk is a porous limestone sometimes containing chert or flint nodules. Reef limestone is formed by corals, i.e. animals living in warm ocean water at moderate depth. A characteristic feature of many sedimentary rocks, particularly sandstone and shale, is a layered structure. The stratification is a result of variation in the depositional process.

Engineering properties

Hard sandstone and hard limestone are frequently used as aggregate in road pavements, but most sedimentary rocks are soft and only suitable for embankment construction. Hard sandstone has a good polished stone value but in most cases a substandard abrasion value. The affinity for bitumen varies. Hard limestone may have a high abrasion value and can then be used in subbase and base courses. Limestone bonds well to bitumen. However, most limestones have a very low polished stone value, and asphalt surfacings made with limestone aggregates normally become slippery when wet. Hard limestone is an excellent aggregate for cement concrete because of its low thermal expansion.

Limestone is widespread in many countries in the Middle East and may be the only material available to use as crushed rock aggregate. In other tropical regions reef limestone is very common. In the South Pacific and Indian Oceans many of the islands are formed entirely by corals. Several islands in the Caribbean and parts of the land mass surrounding the Bay of Mexico are formed by reef limestone raised from the sea bed. Some of this limestone is hard and can be used for pavement construction (ref. 2).

Calcrete and silcrete

Calcrete and silcrete are chemical sediments produced by carbonate-holding groundwater in particular localities in deserts and semi-deserts. The water is brought to the surface by capillary action where evaporation produces a hard precipitate. The composition of the deposits depends on the type of the underlying rock. Carbonates of calcium and magnesium are redeposited as calcrete while silcrete is formed by siliceous precipitates. Massive calcrete resembles soft limestone while silcrete is similar to soft sandstone. Both calcrete and silcrete may also occur as nodular gravels. Calcrete and silcrete are normally only used for road making in areas where stronger materials are not available. Both materials may be stabilized with cement.

Gatch

'Gatch' is a calcified sand known from Kuwait. Because good granular materials are in short supply in Kuwait, it is necessary to use crushed gatch as a road base material. However, crushing and breaking of the cementation bonds may transform gatch into a swelling granular material that absorbs water when it becomes available resulting in cracking and damage to the pavement. The swelling potential may be reduced by use of cement or lime stabilization (ref. 3).

13.2.3 Metamorphic rocks

The principal metamorphic rocks are quartzite, marble, slate, schist and gneiss. The extent of metamorphic rocks is rather limited.

Quartzite, marble, slate, schist, gneiss

Metamorphism of sandstone and limestone produces quartzite and marble respectively. Shale is altered to slate and schist. Slate and schist have a pronounced laminar structure. At the highest metamorphic grades the folia become less distinct and the grain size coarser. This rock is called gneiss and resembles coarse-grained granite.

Engineering properties

Quartzite usually makes decent road aggregate. The abrasion and impact values are good and the polished stone value may be high. The affinity for bitumen is variable. Marble has properties similar to those of hard limestone. Gneiss has properties similar to those of granite. The foliated metamorphic rocks, slate and schist, have very poor crushing strengths parallel to the banding and can only be used as a fill material.

13.3 SOILS

There are three different soil-forming processes. Residual soils are formed in place by weathering of bedrocks. Sediments are formed from parent materials that have been transported to their location by wind, water or glacial ice. Organic soils are formed from decomposed plant and animal materials.

The variation of tropical soils is extreme. Six of the most widespread groups of tropical soils are described in the following. No organic soils are included. Organic soils create serious problems for road engineers in many temperate regions but are rare in the tropics except in areas that are perpetually flooded. In hot, wet climates decomposed organic matters are soon leached out of the soils.

13.3.1 Laterite

Laterite is a group of highly weathered soils formed by the concentration of hydrated oxides of iron and aluminium (ref. 4). The iron oxides produce yellow, ochre, red or purple colours. Red is normally predominant. Other names for lateritic soils are roterde, ochrosols, ironstone (Nigeria), ferricrete (Southern Africa), mantle rock (Ghana), and murram (East Africa).

The term laterite may correctly be applied to clays, sands, and gravels in various combinations. However, a pronounced tendency to call all red tropical soils laterite has caused much confusion.

Distribution

Laterite is a very widespread soil group. Lateritic soils occur in all wet tropical regions, e.g. East, West and Central Africa, Indonesia, Thailand, Brazil and various islands such as Hawaii and Cuba (Figure 13.1).

Formation

Lateritic soils are residual soils formed in hot, wet tropical regions with an annual rainfall between 750 and 3000 mm or more. The main soil-forming

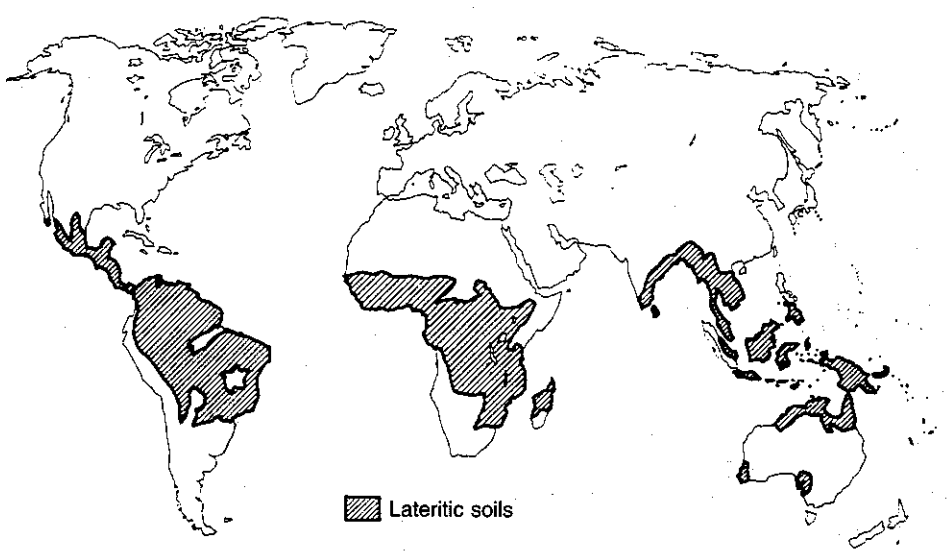


Figure 13.1 Distribution of lateritic soils (ref. 4).

process consists of intensive weathering with leaching of bases and silica resulting in a relative accumulation of iron and aluminium oxides and formation of kaolinitic clays. Intensive weathering producing deep laterite profiles occurs on flat slopes in the terrain where the runoff is limited. On level ground, where drainage is poor, expansive clays dominate at the expense of laterites. Expansive clays are discussed in a later section.

Two aspects of the parent rock affect the formation of laterite. One is the availability of iron and aluminium minerals. These are more readily available in basic rocks. The other is the quartz content of the parent rock. Where quartz is a substantial component of the original rock, it may remain in the weathering product as quartz grains.

Hardening

If the iron enriched laterite is dehydrated, hardening or concretionary development takes place. Dehydration may be caused by climatic changes or upheaval of land. Dehydration may also be induced by human activities, e.g. clearing of forest.

Laterite mainly occurs as:

- surface deposits of unhardened, clayey soils;
- massive rock-like hardpans;
- gravel consisting of concretionary nodules in a soil matrix.

Unhardened clay

Unhardened, clayey laterite is widespread in Southern Asia. It is this soil that has given the name to laterite, meaning 'brick' in Latin. The first reference is from India, where this soft, moist soil was cut into blocks of brick size and then dried in the sun. The blocks became irreversibly hard by drying and were used as building bricks. Blocks cut from laterite were used to construct the famous now-deserted city of Angkor in Cambodia.

Hardpans

Rock-like crusts of hardened laterite occur in Southern Asia, Australia, and Africa. Because of high resistance to erosion, these so-called hardpans are often found as cappings on detached plateau remnants (mesas), but their formation would have been in lower parts of the slopes. The hardpans can be subjected to a new cycle of weathering and transportation in which hardpan fragments are redeposited elsewhere as secondary deposits. Rock-like laterite is usually very heavy and abrasive and unsuitable for quarrying and processing to road aggregate.

Gravel

Lateritic gravels occur in almost all tropical countries. They consist of gravel-sized concretionary nodules in a matrix of silt and clay. The resulting particle size distribution is gap-graded (Figure 13.2). Lateritic gravel deposits often stand out as low humps in the terrain. Typically, they take up an area of several hectares and a thickness of between 1 and 5 metres.

Vegetation

The natural vegetation of lateritic soils in regions with high rainfall is dense, tropical rainforest. When the soils are cultivated, the organic matter is mineralized rapidly, and soil fertility becomes extremely low. In regions with lower rainfall lateritic soils mainly form bush savannah. Fertility is low because the hardened soil restricts growth.

Engineering properties

In the tropics, where weathering is often intense, the availability of suitable rock as a source of crushed aggregate is often limited. In these areas lateritic gravel is a traditional source of road aggregate. When the grading of lateritic gravel is close to a mechanically stable particle size distribution, the material performs satisfactorily on lightly trafficked roads, both as a subbase and a base under thin asphalt surfacings, and as natural gravel surfacings. But often the significant silt and clay

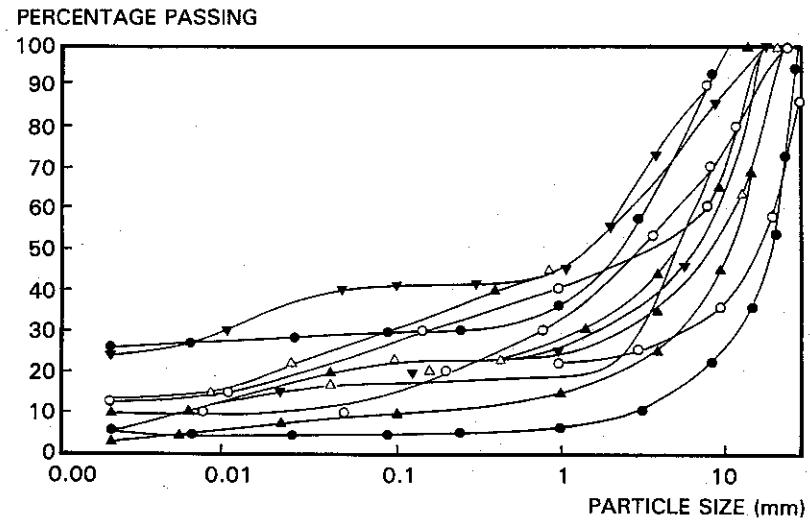


Figure 13.2 Typical gradings of lateritic gravels (ref. 2). (Crown copyright; reproduced with the permission of the Controller of HMSO.)

content renders the material moisture sensitive. The nodules are often hard with a good abrasion value, but in some deposits the nodules are so weak that they can be crushed with the fingers. Many laterites contain a proportion of quartz. If the nodules are comparatively weak, the strength and durability of the coarser fraction become a function of the proportion of quartz. The widespread evidence of the deterioration of laterite roads emphasizes the need for careful assessment of lateritic gravel to be used for road construction.

Self-hardening

According to ref. (5) some deposits of lateritic gravel are immature and exhibit self-hardening properties when drying. A tendency to self-hardening may advocate relaxation of the traditional requirements governing the selection of road materials. Gravel, which is normally considered mechanically unstable and too plastic, may be satisfactory when built into the pavement. According to ref. (2) the hardening of laterite pavements is rather due to a combination of other causes. The compaction is improved by the traffic. Weak, coarse particles tend to fracture both under construction and traffic, and this somewhat alleviates the gap grading and helps improve the stability.

Stabilization

The tendency for laterite gravels to be gap-graded with depleted sand-fraction, to contain a variable quantity of fines, and to have coarse particles of variable strength which may break down, limits their usefulness as pavement materials on roads with heavy traffic. For heavy duty pavements their performance characteristics need to be improved by appropriate stabilization measures. Stabilization of laterites has proved successful with both cement and lime. In particular, satisfactory results with cement have been reported.

Asphalt

Many cases have been reported of the failure of new asphalt pavements placed on old laterite roads. The failures may be due not only to the moisture sensitivity of the laterite, but also to detrimental vapour pressures that may develop under the asphalt if the gravel base is very dense and humid, and the sun is shining on the black, heat-absorbing road surface.

Gradation analyses

Gradation analyses of laterite require attention. It is important that the sample preparation and test procedure do not fracture the particles. Furthermore, grading should be calculated by volume proportions as well as by mass proportion as the coarse, iron-rich fraction in laterite usually has a specific gravity of 3.0–3.5 while the fine, kaolinite fraction has a specific gravity of about 2.7.

Compaction tests

Breakdown of weak particles may affect the compaction characteristics, and fresh material should be used for each point on the compaction curve.

Particle strength

The Los Angeles abrasion test is commonly applied for evaluation of the particle strength. For lateritic gravels, however, the water absorption test may be used as a low-cost alternative. The water absorption is fairly consistently related to the abrasion value as shown in Figure 13.3.

Plasticity tests

The plasticity limits of laterite can be misleading if it is not realized that a high proportion of hydrated oxides can change the properties of the fines. The oxides tend to coat the surface of the individual soil particles. The coating can reduce the ability of the clay mineral to absorb water. It can also cause a cementation of the adjacent grains. Both factors reduce plasticity, but intense remoulding of the soil breaks down the oxide coatings and the aggregations, with an attendant increase in the plasticity.

When soils containing hydrated oxides of iron and aluminium are dried, they may become less plastic. The process cannot be reversed by re-wetting. The effect is particularly pronounced when the material is oven-dried.

The susceptibility of lateritic soils to the effect of clay mineral aggregation and to drying should be identified by determination of the plasticity limits after different mixing times and by comparing the plasticity limits of soil prepared from natural

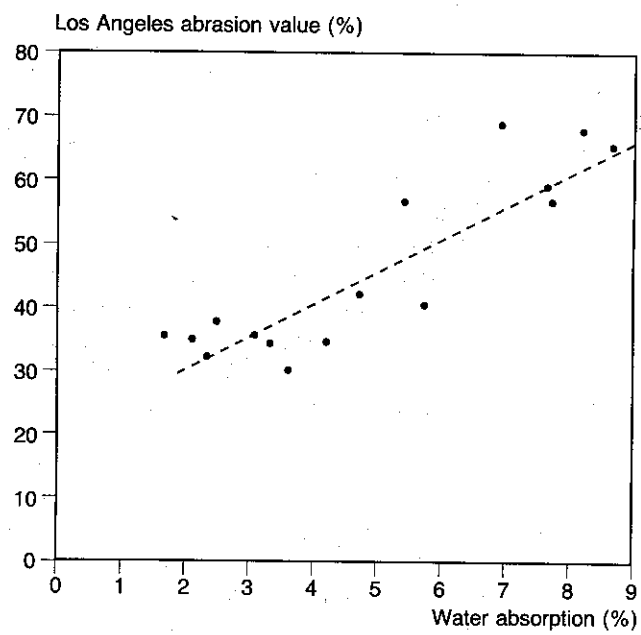


Figure 13.3 Relation between water absorption and Los Angeles abrasion value for typical West African lateritic gravels (ref. 4).

moisture content with those of oven-dried samples rewetted to the point of test, cf. Chapter 12.

13.3.2 Desert soils

Desert soils are normally called arid and semi-arid soils by geologists. Deserts are defined as regions with less than 100 mm average precipitation and semi-deserts as regions with less than 300–400 mm on an annual basis. However, the total precipitation is typically very irregular from year to year, between seasons of the year and from place to place. Rain often falls in short heavy, local showers.

Distribution

Desert soils mainly occur in the subtropics although there are also dry regions in the tropics. More than 30% of the land surface of the earth consists of deserts and semi-deserts. They include the Kalahari, Namibia, the Sahara, Somalia, Arabia, Iran, Turkestan, Takla-Makan, the Gobi, Patagonia, the Atacama, the main part of Australia and part of North America. Dry regions are expanding in the developing countries by 6 million hectares annually. The advance of deserts is by many specialists considered as the biggest environmental problem on earth.

Formation

The soil-forming process in dry regions is essentially the same as that in humid regions. However, because of the limited amount of water available in the soil profile, the reactions are relatively less intense. As a result the soils inherit much of their composition from the parent material.

Characteristics

The air is extremely dry in deserts and semi-deserts. When it is raining, much water is lost by evaporation and runoff. If there is no influence of groundwater, then the lower part of the soil is always dry. Especially in real desert soils, infiltration of rainwater is restricted because the wetting and rapid drying of the soil surface create a thin, hardly permeable crust on the surface. Therefore, most rainwater is lost as runoff causing sheet and gully erosion. The runoff finally enters deep wadis through which an enormous amount of water flows after a heavy shower. Common soil types in deserts and semi-deserts are sand and saline soils.

Sand

Erosion and transport by wind are common in desert regions, which lack a protective cover of vegetation and humidity to bind the soil grains together. If the original soil was gravelly, a layer of gravel, pebbles and stones is left behind on the surface forming a 'desert pavement' that protects the soil from further erosion. In other places wind-blown material is accumulated on top of other soils. Sometimes it forms a thin sheet. In other places the wind-blown material forms drifts or dunes. Sand-dunes differ from sand-drifts in that no fixed obstacles are necessary to initiate the formation of sand-dunes. About 15% of the surface of the Sahara is covered by drifting sand-dunes. Other large bodies of drifting sand are found in Saudi Arabia. The material forming desert drifts and dunes is normally fine quartz sand.

Saline soils

In low-lying areas, particularly on the coast, saline groundwater may be brought to the surface by capillary action. Evaporation causes the salt to accumulate in the upper layer of the soil. The types, mixture and concentration of salt vary. Sodium chloride is often less than half the total, which may also include sodium sulphate and chlorides of calcium and magnesium. Nitrates and borates rarely occur. The salts may occur as salt crystals, salt crusts and salic horizons. The soils have mostly a loose porous granular structure. If hygroscopic salts such as calcium and magnesium chlorides are present, they attract moisture from the air making the soil

look moist in the morning. If salt reaches the surface a salt crust may form. Saline soils are common in some deserts, particularly in the Sahara and Saudi Arabia.

Sabkha

Some saline soils also contain carbonates. The sabkha known from the coastal areas of the Arabian Peninsular is a saline soil containing concretionary carbonate.

Vegetation

The natural vegetation on non-saline desert soils is scanty grass growth and small bushes. Overgrazing and cultivation of soils that are too dry have caused severe wind erosion in many desert regions. Saline soils are supporting very little vegetation. Irrigation of arid and semi-arid soils involves a risk of salinization so that the land is damaged and made unproductive.

Engineering properties

Desert sands usually occur at low field densities. When constructing roads on loose sand, it is necessary to compact the sand thoroughly in order to avoid uneven settlement of the road. However, it may be difficult to obtain satisfactory compaction, because water is scarce and dry compaction is not very effective with the often single-sized soil material. The solution may be to use vibratory compaction to obtain in-depth compaction followed by static compaction to settle the top layer.

Drift control

When locating roads in desert regions the biggest problem is to avoid drifting sand. In principle drifting sand should be controlled in the same way as drifting snow is controlled in regions with a cold climate.

Road cuttings in tracts of blown sand should be avoided as they act as sand traps. The road should be placed in the windward side of obstacles so that sand is blown clear of the road. The cross-section of the road should be raised slightly and made as smooth as possible. Side drains should be avoided if possible. In some places establishment of vegetation has been tried as a means to combat sand drift.

Stabilizing

Because of the single-sized particle distribution desert sands are usually not very suitable for stabilizing with cement, but they can often be effectively treated with bitumen.

Use of saline soils

Salt-holding sand and sabkhas can be used as a fill and road base material if special precautions are taken. Hygroscopic salts in soils have a stabilizing effect. If the salt content is high, however, the stability is drastically reduced when the soil takes up water. Therefore, embankments and bases made from saline materials should normally be protected from being soaked during rain.

Asphalt

Salts have no destructive chemical effect on asphalt, but a high salt concentration in the pavement, subgrade and/or groundwater may result in blistering of thin asphalt pavements. The blistering is caused by salts migrating upwards and crystallizing below the asphalt. The salt is precipitated from evaporating capillary moisture. Thicker asphalt pavements (30 mm or more) do not appear to be affected by salt. They are more resistant to high evaporation rates and crystal pressures (ref. 6).

Concrete

Sand and gravel contaminated with salts should normally be avoided as an aggregate in cement concrete. Sulphates are especially detrimental. In the Middle East, several concrete structures of more recent date have disintegrated because of sulphates in the aggregates. Concrete structures covered by saline soil, e.g. culverts and bridge foundations, should be made of concrete with low permeability and protected from seeping groundwater.

Testing

There are no special difficulties in performing classification and engineering tests on desert soils. Salt contents may be determined by use of traditional titration tests.

13.3.3 Expansive clays

The most well-known example of expansive clay is black cotton soil. This name is believed to have originated from India where locations of these soils are favourable for growing cotton. Many other names are applied locally, such as margalitic soils in Indonesia, black turfs in Africa, and tirs in Morocco. Expansive clays swell when moistened and shrink when dried. The swelling properties are due to a high content of the clay mineral montmorillonite. The colour is black, dark grey, or dark grey-brown. The dark colour comes from a thorough mixing of a little organic matter with the clay.

Distribution

Expansive clays occur in a great many tropical and subtropical countries. The most extensive regions with expansive clays are found in India, Australia and Sudan. Less extensive areas occur in almost all tropical and subtropical countries especially in Africa (Figure 13.4).

Formation

The parent material of most expansive clays is transported material, although some types are formed in weathered material from basalt or limestone *in situ*. The climate should be warm with alternating dry and wet seasons and an annual rainfall ranging from 300 to 1000 mm. Most expansive clays form on poorly drained areas. Expansive clays rarely develop thick profiles. Often the layer thickness varies between 1 and 2 metres only (ref. 7).

Characteristics

During the dry season cracks develop in the surface of expansive clays because of the shrinkage. The cracks may be 1 cm wide and more than 50–80 cm deep. The cracks open up in a polygonal pattern, each polygon having a diameter of 1–4 metres. The higher the clay content, the smaller the polygons. During the dry season small particles from the surface fall into the cracks. During the wet season when the soil swells, the partly filled cracks cannot close and subsoil is pressed to the surface. In this way small mounds are formed, and former cracks become small depressions. Many expansive clays have a loose granular surface due to the cracking and swelling in the upper few centimetres of the surface. This process is called self-munching.

Vegetation

The natural vegetation on expansive clays is bush or grass savannah. In many countries expansive clays are important agricultural soils because of their high fertility. However, the soils are difficult to cultivate because they are very hard in the dry season and very sticky in the wet season.

Bearing capacity

The CBR of swelling soils is very dependent on the water content. This may be illustrated by an example from Africa where an asphalt road was constructed on an expansive clay subgrade. After three months of dry season the registered water content was 20% in the subgrade below the asphalt and 8% below the unsealed shoulders. The corresponding CBR values were 30% and 3%, respectively.

Swelling

Swelling of expansive clay depends on the content of swelling clay mineral (montmorillonite), the initial moisture content and the amount of water the clay is allowed or able to absorb. It is difficult to evaluate the potential swelling by simple laboratory testing as the results are very sensitive to the test conditions. If an expansive clay is allowed to absorb water and at the same time prevented from expansion, a high swelling pressure develops. If a small expansion is allowed, the expansion pressure is reduced considerably.

Plasticity

The liquid limit of most expansive clays is over 50%. The plasticity index normally falls between 20 and 60%. The liquid limit related to the clay content

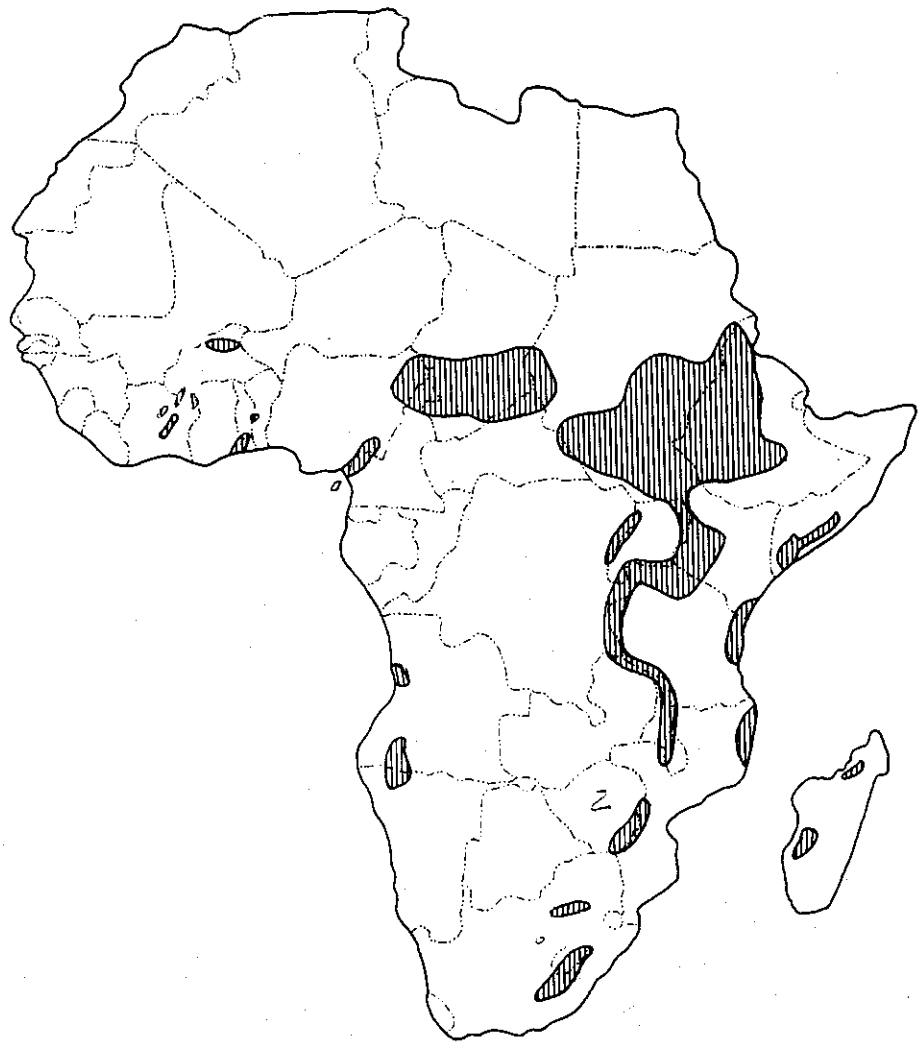


Figure 13.4 Distribution of expansive clays in Africa (ref. 5).

may be used as an indicator of the swelling potential (Figure 13.5). A high liquid limit, however, is not a proof of expansiveness. Also non-expansive clays may have a high liquid limit.

Free swell

The free swell test is a simple method to verify swelling tendencies, cf. Chapter 12. It provides no quantitative measure of either pressure or volume change, but nevertheless it is useful in identifying problem soils. The test results on a number of expansive clays from Africa varied between 50 and 100%.

Engineering properties

Expansive clays with a high swelling potential are a persistent problem in highway construction in tropical regions with pronounced dry and wet seasons. Road construction alters the moisture pattern in the subgrade soil. Surface evaporation is reduced by the road pavement and, after the road has been completed, the

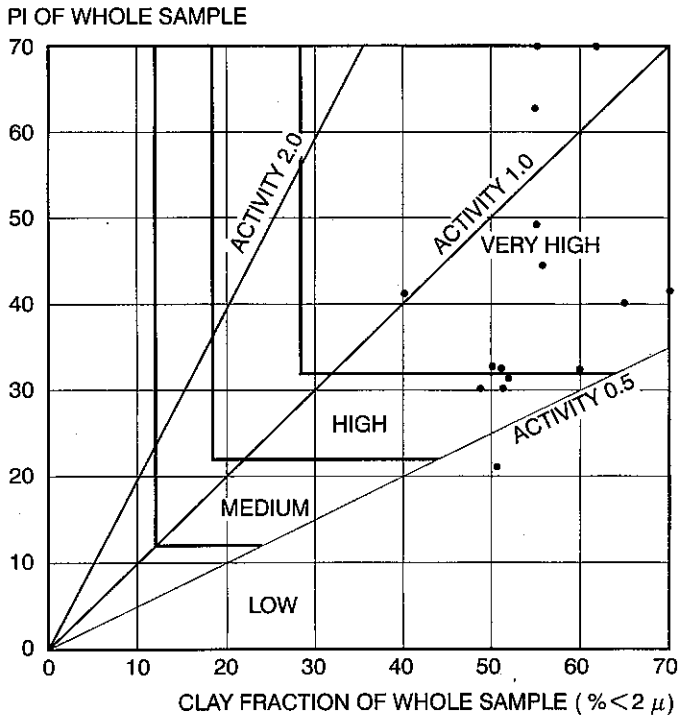


Figure 13.5 Potential swelling tendency of clays (ref. 5).

moisture content of the subgrade normally rises. This causes swelling of the subgrade and heaving of the pavement. The swelling also reduces the bearing capacity of the soil. The heaving of the pavement may be considerable if the pavement is placed when the subgrade soil is in a dry state. During the following wet and dry seasons the surface of the road will move up and down depending on the moisture changes. The seasonal movements will be less pronounced along the centre of the road than near the shoulders and the side drains where water is able to infiltrate and evaporate. In highly expansive clays the yearly upward and downward movements of the pavement edges may be as high as 5–10 cm causing severe edge failures.

Replacement

There are no inexpensive means of utilizing highly expansive clays satisfactorily. The obvious solution is to avoid them wherever possible, or if shallow, to excavate and waste the material. However, in many areas where these soils occur, the deposits cover such large areas that avoiding or bypassing them is not feasible, and there are no materials suitable for fill.

Thick base

Roads over expansive clays require a great thickness of base and subbase, as dictated by low CBR values even in the absence of swelling.

Drainage

Drainage is extremely important. Poor drainage results in big seasonal variations in the subgrade moisture. However, if ditches are too deep, or too close to the pavement structure, seasonal drying or partial desiccation may be pronounced along the shoulders. The right solution is to locate the ditches at some distance from the pavement and seal the shoulders.

Prewetting — The heave of new pavements may be reduced by prewetting of the subgrade before placing of the pavement. Prewetting has been used with varying success.

Embankment — The seasonal expansion of the subgrade may be minimized and smoothed out by confining the subgrade under an embankment. However, this approach assumes that suitable embankment material is readily available.

Lime stabilization — Expansive clays are normally very suitable for lime stabilization. Lime stabilization increases bearing capacity and reduces the potential swelling in the top layer of the subgrade. The treated layer is more impermeable so moisture variations are reduced. Several cases are reported where an admixture of 4–6% hydrated lime has made an expansive clay suitable as support for a traditional road pavement.

13.3.4 Volcanic ash soils

Names — Volcanic ash soils are residual soils formed in tropical regions with current or recent volcanic activity. In Indonesia these soils are known as asgronden or zwarte stofgronden.

Distribution — Volcanic ash soils occupy large areas in Central America, western South America, Hawaii, New Zealand, Japan, the Philippines, Indonesia, some East African countries and Cameroon.

Formation — When volcanic ash weathers it retains much of its original loose structure. In hot humid climates the weathering forms highly plastic clay containing allophane, an amorphic mineral. In regions with constant high humidity the allophane stage can persist for long periods. In climates with distinct dry seasons the allophane stage is followed by development of the clay mineral halloysite (ref. 2). While the particle shape of other clay minerals is flat and plate-like the particles in halloysite are curled and twisted.

Characteristics — Allophanic clays have a very fragile structure that may collapse when loaded. The presence of allophane and the porosity of the pumice-like parent material cause a high water-holding capacity. The natural water content may be more than 200%. If allophanic clay is heated, the small clay-sized particles develop aggregations, and the plastic clay changes into a non-plastic, sandy soil.

In halloysitic soils the curled clay particles help create an open structure. The structure may be reinforced by precipitated iron oxides. A typical example is the red-coffee soil occurring in East Africa. These soils are free-draining, and the structure is much stronger than in allophanic clays. Heavy working, however, may break down their structure irreversibly.

Vegetation — Volcanic ash soils have a high natural fertility and they are among the best agricultural soils in the tropics. They support different types of crops.

Engineering properties — Allophanic clays are highly sensitive to disturbance. These soils should be avoided by the road engineer. Care is even needed when working in their vicinity.

Roads can be constructed over halloysitic clays but heavy compaction of the soil should be avoided as this may break down the soil structure and render the soil weaker and more susceptible to the effect of water. Studies (ref. 9) have demonstrated the potential for line modification of halloysitic clays.

Testing — Halloysitic clays can change irreversibly to metahalloysite upon drying. Sieving analysis, plasticity tests and compaction tests are likely to break down the structure in halloysite with the effect that the test results do not represent the engineering properties of these soils. For the same reason CBR tests on remoulded samples

may not indicate the true bearing capacity. The compaction properties and the design CBR of halloysitic clay soils should rather be established from field trials or local experience.

13.3.5 Tropical alluvial soils

Alluvial soils are mineral soils that have been transported and deposited by flowing water.

Distribution

Alluvial soils are scattered all over the world. These soils are not specific to the tropics but deserves mention because the coarser types are important sources of road-making materials in many countries.

Big rivers and deltas in the tropics with vast alluvial deposits are the Indus, Ganges, Brahmaputra, Mekong, Irrawaddy, Digul, Congo, Niger, Amazon and Orinoco. Coastal flats with clay sediments are well known from Surinam, Guyana, Sierra Leone and South-East Asia. Irrigational sediments, now forming alluvial soils, are best known from Iraq.

Formation

Alluvial soils mainly occur in:

- river plains and deltas;
- alluvial fans;
- former lake bottoms;
- old irrigated areas;
- coastal plains.

River plains and deltas

River plains and deltas are landforms created by the processes of stream meandering and overbank flooding. Most alluvial soils form flats in the lowest parts of the landscape, and they are normally wet with a high water table, permanently or during specific periods.

River banks are eroded on the outside and downstream side of each curve. On the inside of each curve, sand and gravel are usually deposited. These deposits are called 'point bar deposits'. As the radius of river meanders becomes larger, a point is reached where the river changes its course. The meanders are cut off, and natural basins are formed. When the river is overflowing, its bank material is deposited outside the normal stream channel as natural levees. In deltas the pattern of deposition is similar to that of flood plains, but the rivers have more branches.

Alluvial fans

Alluvial fans are deposited at the base of steep slopes where streams discharge into an area having a more gentle relief. The fans are built of coarse-textured materials eroded from the higher elevations and transported downslope. Alluvial fans are best developed in arid areas at the base of high mountain ranges where they may be several kilometres long. In humid areas fans are smaller, typically only a few hundred metres.

Lake bottoms

The lacustrine pattern of sedimentation is less distinct. Usually soils are uniform and clayey, often mixed with organic material.

Coastal plains

Coastal plains are deposited in sea-water, influenced by the tides. The saline clayey soils are often called marsh soils. Near the coasts, there are former beach-walls with sand, gravel or shells.

Characteristics

Alluvial deposits are normally stratified with a uniform particle size in the individual stratum. The particle size depends on the velocity of water during

depositing. The mineral composition is heterogeneous, reflecting the geology of the parent materials upstream in the catchment area. They may consist of a mixture of various unweathered, weathered and partly weathered minerals depending on the cycles of erosion and redeposition the materials have gone through.

In wet and humid tropical regions, the sediments often consist of almost completely weathered material. In volcanic regions, some fresh minerals may be present. In arid and semi-arid tropical regions, the sediments are normally made up of less weathered minerals, often high in carbonates and gypsum. Marine sediments usually contain weatherable minerals.

Fine-textured clay sediments deposited under water contain much more water than normal soils, and the packing of the particles is very loose. When water drains off or evaporates, there is a gradual vertical shrinkage. On drying, air penetrates the soil through cracks, and the bluish-grey colour changes to brownish by oxidation. Other chemical and biological processes take place. The transformation of the clay is irreversible. An alluvial clay once ripened can be partly reduced by submerging it again, but it never takes up all the lost water.

Vegetation

The weathered alluvial soils from the wet and humid tropics are not as fertile as is usually the case with the less weathered alluvial soils from temperate regions. The special crop is rice, but sugar-cane, bananas, and on better drained sites, cocoa, coffee and citrus are grown, too.

Engineering properties

Good quality road construction materials and aggregates can usually be extracted from point bar deposits in flood plains and from alluvial fans. When surveying gravel and stone deposits, it should be realized that if stratified materials with different grain size in the individual layers are mixed, they may take up a volume 25% less than the combined volumes of the layers. The smaller grains will partly fit into the space between the coarser grains. In order to satisfy the requirements for aggregate to be used for untreated bases, cement concrete or asphalt concrete, it is often necessary to process the materials. The processing may include washing, sieving and/or crushing.

Testing

When investigating the engineering properties of alluvial soils, traditional testing methods are applied, but it should be remembered that the properties of fine-textured clay sediments may change irreversibly when dried and oxidized.

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14 Earthworks and pavements

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14.1 EARTHWORKS

Definitions

Construction of new roads, especially major highways, nearly always involves some earthworks. Earthworks are those construction processes involving soil and rock in its natural form and preceding the building of the road pavement. General aspects of earthwork operations are described in many textbooks, e.g. ref. (1). Some special features of earthworks in tropical rocks and soils are touched on in Road Note 31 (ref. 2) and ref. (3).

Earthwork operations may be classified as:

- clearing and grubbing;
- excavation;
- construction of embankments;
- compaction;
- finishing operations.

14.1.1 Clearing and grubbing

Clearing and grubbing are defined as the removal of trees, stumps, roots, debris, etc. from the area of proposed excavation and embankment. Clearing refers to the removal of material above existing ground surface. Grubbing means the removal of objects to a nominal depth below the surface. On equipment-based road projects clearing and grubbing operations are generally performed by bulldozers with various attachments (Figure 14.1). A considerable amount of hand labour may also be necessary.

14.1.2 Excavation

Excavation is the process of loosening and removing rock or earth from its original position and transporting it to a fill or waste deposit. Excavation is often divided into three categories: roadway and drainage excavation, excavation for structures, and borrow excavation.

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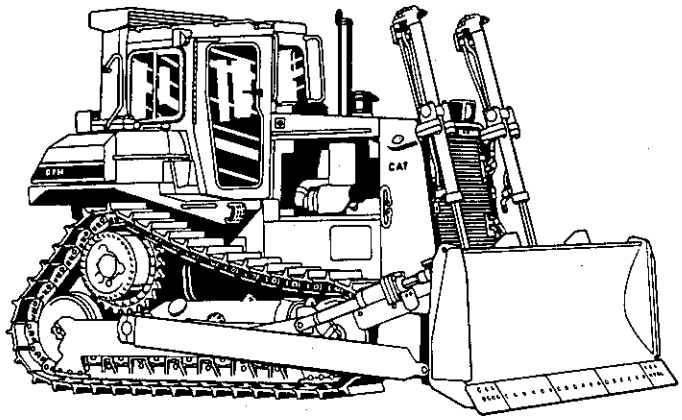


Figure 14.1 Bulldozer.

Roadway and drainage

Roadway and drainage excavation means excavation of the roadway in cuts and the excavation of ditches. Excavated materials capable of being compacted to form a stable fill are used for construction of embankments, subgrades, and shoulders or as backfill for structures. Unsuitable and surplus excavated material is disposed of. Excavated top-soil is usually stockpiled for later use on side slopes in cuts and on embankments. If the soil forming the bottom of a cut is not suitable as the foundation for the road pavement, it may be necessary to remove the soil and replace it with satisfactory material.

Drainage

In order to prevent wetting and softening of the soil it is important that all cuts are kept well drained during the whole excavation operation. Ditch work should be scheduled with this object in view.

Slope stability

Cuts through sound rock can stand at or near vertical. In weathered rock and soil the condition is more unstable. Usually, the side slopes are made steeper than the original ground surface, and slides may occur during construction or at a later date. In some mountainous countries even, many slopes of the natural ground are unstable, and slides are a persistent problem. An example is the Himalayan kingdom of Bhutan, where in 1993 more than 130 slides were 'alive' along the road from the capital Thimphu to Mongar in East Bhutan, a distance of 400 km.

Instability of natural and man-made slopes may be revealed by a range of features:

- springs and patches of reeds in the slope side (indicating that water may cause problems during the rainy season);
- trees leaning at different angles (not trees leaning downslope at the same angle);
- an irregular, pocketed surface with cracks and small ponds or the presence on the slope of hollow bowl-shaped depressions with a steep head;
- weak, weathered and highly fractured rocks;
- rocks with bedding lying parallel to the hillside.

Slides are usually provoked by an accumulation of water in the rock or soil, and slides often occur along planes of weakness. The stability of slopes depends on many factors, the most important being the dimension of the slope, the angle of

the slope, the type of material, and stratification of the rock or soil. Investigations of difficult situations should be left to specialists.

Reducing the risk of slides

The risk of slides in cuts can be reduced by flattening all slopes, but in steep, mountainous terrain this solution may not be practicable. Even in cases where the slopes can be flattened, the most economical solution may be to adjust the slopes only when and where slides occur. If possible, new roads should not be located in areas where slides can be expected.

Equipment

Selection of equipment for excavation and moving of earth depends on the nature of the material, how far it has to be hauled, climatic conditions, the skill and knowledge of the equipment operators and economic considerations. Dozers are normally useful in performing excavations involving very short hauls and for spreading dumped materials. For moderate and longer hauls scrapers push-loaded by tractors may offer lower costs. However, this type of very specialized equipment is rarely seen in developing countries. For hauls of considerable length or over the public highways, trucks loaded by front-end loaders (Figure 14.2) or power shovels may be the cheapest solution. A motor grader (Figure 14.3) may be used for shaping ditches, trimming slopes, and shaping the cross-section of the road. A crane with dragline is frequently used for excavation in swampy land. In the case of rock excavation, drilling and blasting will be necessary, and a crane with shovel may be needed to load the blasted rock into trucks.

Structural excavation

Structural excavation includes the excavation of material in order to permit the construction of culverts, foundations for bridges, retaining walls, etc. Suitable materials taken from structural excavations are used either in backfilling around the completed structure or in other parts of the roadway structure. Both machine

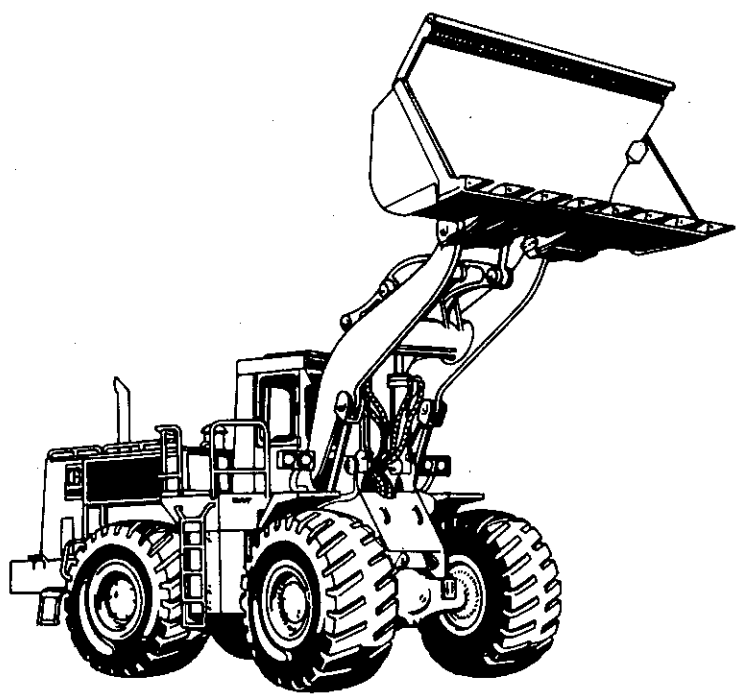


Figure 14.2 Front-end loader.

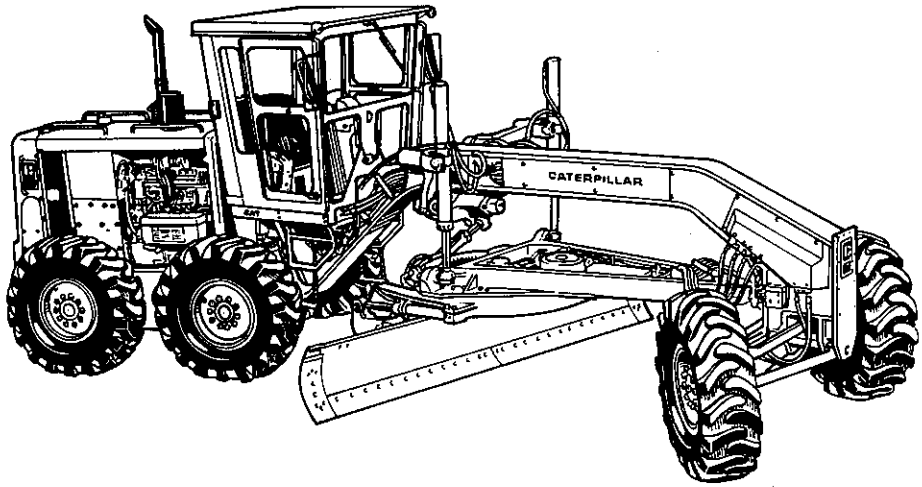


Figure 14.3 Motor grader.

and hand methods are used for structural excavation, but more hand labour is required in this operation than in other types of excavation. Bulldozers can be used to good advantage for the excavation and backfilling of many structures. Front-end loaders also find application in this work. For deep excavation a crane with clamshell is suitable because of its ability to work in vertical lines and in close proximity to bracing and sheeting required in deep excavations.

Borrow excavation

Sometimes sufficient material for the formation of embankments is not available from excavations performed within the limits of the right-of-way. In such cases additional suitable material is taken from borrow pits. Specifications for borrow excavation are generally the same as for roadway excavation. Additional requirements usually relate to the condition which borrow pits should be left in when they are abandoned. Borrow-pit excavation may be effected by use of shovels or front-end loaders and the material loaded and transported on trucks.

14.1.3 Construction of embankments

Embankments are used in road construction when the vertical alignment of the road has to be raised some distance above the level of the existing ground surface in order to satisfy design standards or prevent a damaging effect from surface or groundwater. Many embankments are only 0.5–1.5 m high, but heights of 5 m or more are not unusual on major highways.

Settlements

High embankments impose a heavy load on the underlying foundation soil. On some soil this may result in settlements. If the foundation soil is extremely weak, a slip failure may occur. The residual soils so common in many tropical countries (e.g. laterite) are normally not very compressible. The small settlements that do occur will normally be finished when the embankment is completed. An exception to this is clays developed by weathering of volcanic ash (Chapter 13). Some transported soils are very prone to settlement. Examples are wind-blown sands and alluvial clays including organic marsh soils.

Improving stability

The stability of the foundation for embankments depends on the dimension of the embankment, and the type and thickness of the foundation soil. Expert advice should be obtained when dealing with difficult situations. Methods for improving the structural properties of an embankment foundation include the removal and replacement of unsuitable material. Consolidation (extrusion of water) of saturated, fine-grained soils can be accelerated by use of surcharge and/or vertical sand drains. However, these procedures are very costly, and relocation of the route should always be considered whenever extremely weak soils are encountered at considerable depth.

Side-sloping terrain

The cross-section of a road embankment in flat terrain usually consists of a flat top section with symmetrical side slopes on either side. In side-sloping terrain it is common practice to cut the road partly into the hillside and use the excavated material as fill on the outside of the road. The result is an asymmetric cross-section as illustrated in Figure 14.4.

In steep, side-sloping terrain the embankment is usually keyed to the hillside by cutting horizontal benches into the slope beneath the fill. A retaining wall may be needed to support the fill. The material has to be adequately compacted and well drained.

Suitable fill

Most types of soil and broken rock can be used for construction of embankments, but materials of the AASTHO classifications A-1, A-2-4, A-2-5 and A-3 are to be preferred. More plastic materials may create problems in wet weather. If expansive clays cannot be avoided, special precautions should be taken, as described in Chapter 13. Highly expansive clays and organic soils should not be used as fill.

Slopes

Embankments made from materials of low plasticity can be constructed with slopes as steep as 1:1.5 (vertical : horizontal) without causing slope stability problems. For other soil types, particularly in wet climates, the maximum slope may be 1:2. Where the embankment is subjected to flooding, the slope should not be steeper than 1:3. The side slopes are normally protected from the erosive action of wind and water by establishing a cover of vegetation.

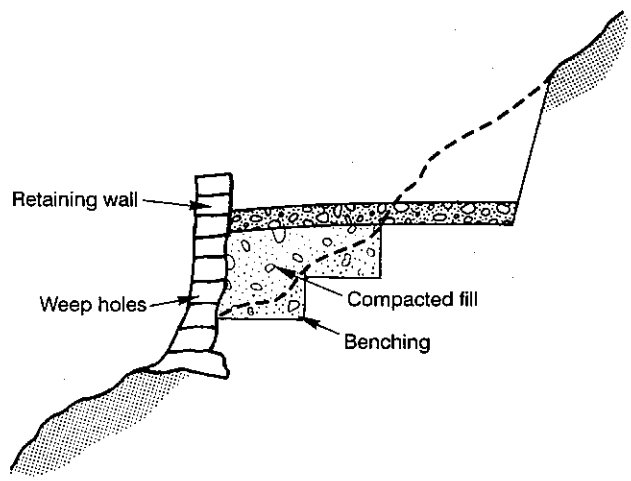


Figure 14.4 Cross-section of a road in steep side-sloping terrain.

Earth embankments are constructed of relatively thin layers of soil. On equipment-based projects the material is usually dumped in the proper location by trucks and spread by dozers or graders. The required maximum thickness of loose soil is usually 25–30 cm. The soil is thoroughly compacted before the next layer is placed. During the construction operation the embankment should at all times be kept well drained especially when material of high plasticity is used. Construction

14.1.4 Compaction

Compaction is defined as a procedure that increases the density of a particular material by expelling air from the voids in the material and thereby bringing the particles into more intimate contact with each other. Compaction is the cheapest and simplest method for improving the shearing resistance of the soil and minimizing future settlements. Therefore, soils in embankments and subgrades in cuts are usually always compacted using special compacting equipment (rollers, vibrators, tampers, etc.). Definition

The result of a compaction work depends primarily on the moisture content of the soil, the type of the soil, the compaction equipment used, and the energy applied.

When a soil is compacted by use of a specific compaction method and energy, the moisture content strongly influences the result. For most soil types, the maximum dry density is achieved at a particular moisture content, the so-called optimum moisture content (Chapter 12). Because of the moisture–density relation, water must be added to dry soils and overly wet soils must be aerated before compaction. However, in dry areas, where it is difficult to provide the large amount of water needed to bring the water content of a dry soil to the optimum level, it may be better to compact the soil in the dry state rather than adding an insufficient amount of water to the soil. In rainy weather, it may be necessary to replace an overly wet soil with more suitable material or stabilize the wet soil with lime, as described later in this chapter. Moisture

The greater the permeability, the easier it is to expel air from the soil. This partly explains why gravel and sand can usually be compacted to a greater density than clay. In well-graded soils, the smaller particles may fill some of the voids between the larger particles. Well-graded soils, therefore, can achieve a greater density than uniformly graded soils. Soil type

The choice of compaction equipment is wide. Most equipment is available in several sizes (deadweights). Compactors may be self-propelled or pulled by a tractor. Equipment

The most common type of compactor is the self-propelled, smooth steel-wheeled roller. The slow-moving roller acts on the soil with 'static' loads. It can be used for compacting all types of soil, i.e. gravel and sand as well as clay.

The steel-wheeled roller may be equipped with devices for vibrating the wheels. The vibration imposes pulsating stresses in the soil. This reduces the friction between the soil particles and results in a highly effective and deep compaction, particularly in gravel and sand. If the vibration is turned off, the roller can be used as a traditional, static roller, a fact which makes the vibratory, steel-wheeled roller (Figure 14.5) a very versatile item of compaction equipment.

Other types of compactor include the pneumatic-tyred roller and the sheepsfoot Other types

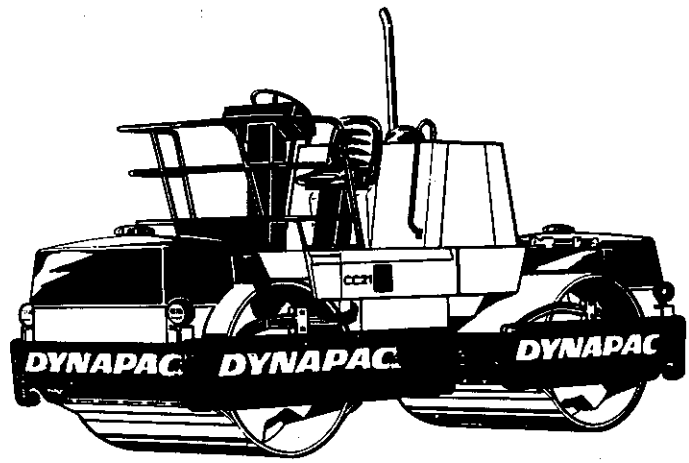


Figure 14.5 Self-propelled, vibratory, smooth steel-wheeled roller.

roller. A pneumatic-tyred roller consists of rubber-tyred wheels mounted on an articulated frame which provides a uniform load on each wheel. A sheepsfoot roller is made of a steel drum to which protruding, tamping feet have been attached. In developing countries pneumatic-tyred rollers and sheepsfoot rollers are only seen on very big road construction projects.

For compaction of backfill in narrow trenches and excavations for structures, a wide variety of hand-operated mechanical tampers and plate-type vibrators is available.

Energy

It is possible to compact a soil to a higher density and to a greater depth by increasing the loading per unit width of the roller (Figure 14.6). However, the loading must not exceed a certain limit depending on the soil type. If the roller produces shear stresses exceeding the shear strength of the soil, the soil will be remoulded and loosened.

As seen from Figure 14.6, in most cases repetitions of roller passes are also effective only up to a certain limit. The compaction effect usually fades out when the number of passes exceeds 8–16 depending on the soil type.

Compaction requirements

Compaction requirements are commonly specified by use of an end product specification. Compaction tests on the particular soil are carried out in the laboratory, and the result is used to define the required density. During construction the densities obtained in the field are determined and compared to the required density. The most common methods applied for measuring field densities are the sand cone and the nuclear density gauge.

It is recommended that the top of an embankment as well as the upper 50 cm of the subgrade in cuts are compacted to a minimum of 95% of the maximum dry density obtained in the modified Proctor test. For the lower layers of an embankment the requirement is reduced to 93%. For coarsely grained soil material the Proctor test should be replaced by vibro compaction.

Volume change

Most soil materials change volume when compacted. Rock fill will occupy 1.2–1.5 times more space than the solid rock. Excavated earth will expand in the transporting vehicle but when compacted it will usually shrink to 0.6–0.8 times the

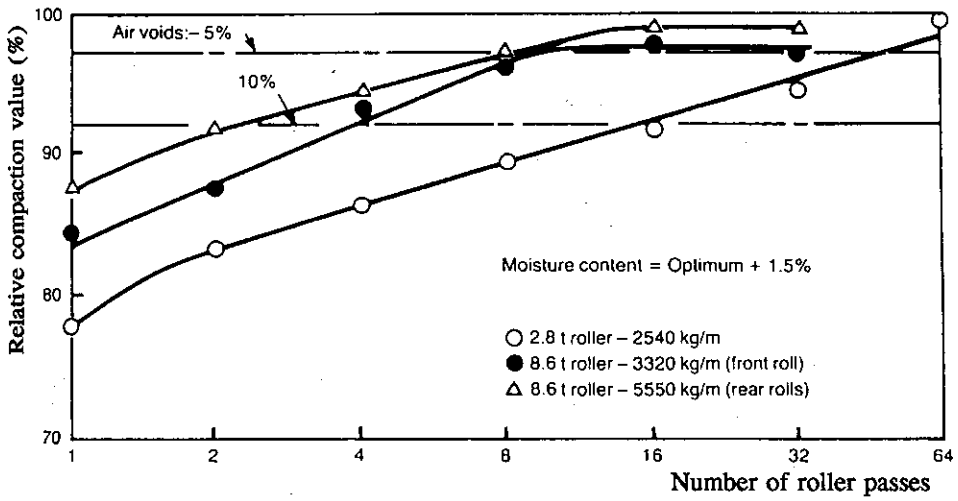


Figure 14.6 Relations between relative compaction of a sandy clay and the number of passes of smooth steel-wheeled rollers (ref. 4). (Crown copyright; reproduced with the permission of the Controller of HMSO.)

original volume depending on the soil type. It is important to take this volume change into consideration when computing excavation and fill quantities.

14.1.5 Finishing operations

Finishing operations are the final activities necessary to complete the earthwork, i.e. trimming of formation level, shoulders, ditches, and side slopes. Most finishing operations are carried out concurrently with other earthwork operations and performed as the job approaches completion. The equipment most widely used for finishing is the motor grader and the dozer.

14.2 PAVEMENT STRUCTURE

A highway pavement is a structure whose primary aim is to support the traffic loads and transmit them to the basement soil after reducing the stresses to a level below the supporting capacity of the soil. Most pavements consist of three superimposed layers each performing different primary functions. The traffic-induced stresses and the influence from weather decrease with the distance from the surface of the pavement. So do the quality requirements and the costs of the pavement materials. The terms used in this book for the different layers in a highway pavement are shown in Figure 14.7.

Wearing course

The wearing course is the uppermost layer of a sealed pavement. It provides the riding surface for the road users. The wearing course should be smooth and dust-free and have adequate skid resistance. It should protect the pavement against wear from the traffic and soaking from rainwater. The most common materials for wearing courses are surface dressing and asphalt-concrete. Earth roads and gravel roads have by definition no wearing course.

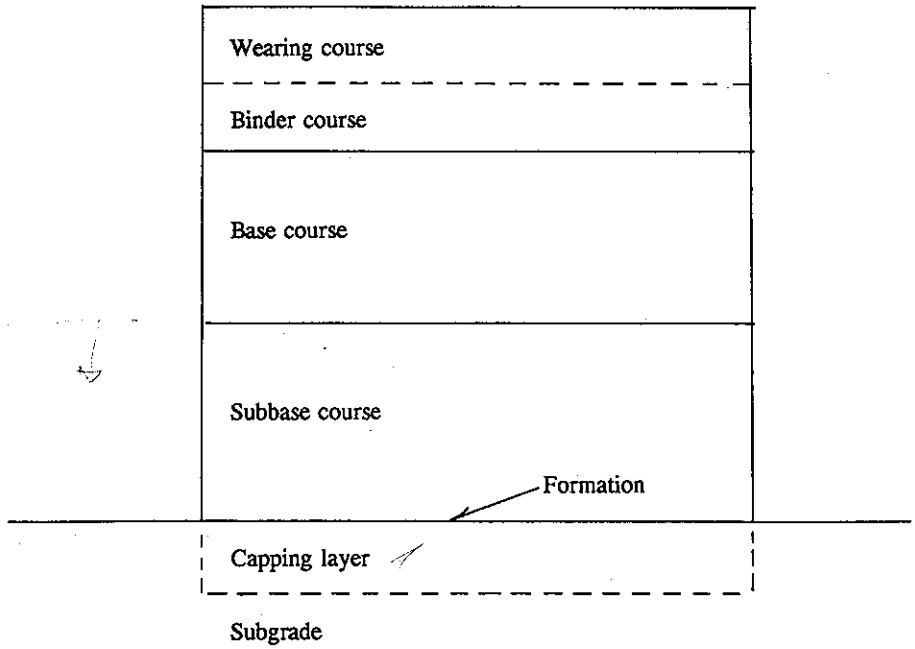


Figure 14.7 Terms used for the different pavement layers.

Binder course — A premixed asphalt surfacing is sometimes laid in two layers. The lower layer of such a construction is known as the binder course.

Base course — The base course is the main load-spreading layer. In order to make sure that the loads applied at the edge of the pavement will be supported by underlying layers the base course is constructed some distance beyond the edge of the wearing course. The base course may consist of premixed asphalt, cement concrete, graded granular gravel, crushed rock, macadam, or materials stabilized with lime or cement. Cement concrete roads combine the base and the wearing course into one and the same layer.

Subbase course — The subbase course is a secondary load-spreading layer. It acts as a separation between the subgrade and the base and as a working platform during the construction of the upper layers in the pavement. When the subbase is made from unbound materials, it may also function as a filter and drainage layer. The subbase is usually constructed from natural gravel or from materials stabilized with cement or lime. Many pavements have no subbase layer as the base course is placed directly on the foundation.

Subgrade — The subgrade is the soil acting as a foundation for the pavement. The subgrade is the result of the earthwork, and it may consist of the undisturbed, local soil or material excavated elsewhere and placed as fill. The surface of the subgrade is called the formation.

Capping layer — A capping layer is a strengthening layer that may be used on very weak subgrades. It is normally defined as part of the subgrade. It is made from imported, selected fill or from subgrade material stabilized with lime.

UK — It should be noted that a different and rather confusing terminology is used in

the UK. The binder course and base course are called 'base' and 'roadbase' respectively.

The design and construction of the most common types of unbound and stabilized pavement layers are described in the following sections. Asphalt pavements are discussed in a separate chapter. Cement concrete pavement is not taken up as this type of pavement is rarely used in developing countries.

14.3 UNBOUND PAVEMENT LAYERS

Unbound pavement layers include capping layers, subbase courses and base courses constructed from gravel and stone materials without addition of a binder. Road Note 31 (ref. 2) gives guidance on the construction of these layers and the criteria that should be met by the materials.

Construction

Unbound pavement materials (except dry-vibrated macadam) are spread by hand or by use of a dozer and/or a motor grader. The material is then compacted to specified density using static or vibrating steel-wheeled rollers. In order to achieve the specified density the water content must be kept as close as possible to the optimum for compaction with the particular roller in use. In dry areas, where water is scarce, dry compaction may be applied.

14.3.1 Capping layer

CBR

Unbound capping layers are normally made from gravelly soils. A minimum CBR of 15% is recommended for material compacted to specified density, usually 95% of the maximum dry density obtained with modified (heavy) Proctor compaction. The samples should be tested at the highest water content expected to occur in the field.

14.3.2 Subbase

Grading requirements

Unbound subbases are generally made from naturally occurring gravel. Screening may be necessary to remove larger-size particles. Recommendations for the particle size distribution are given in Table 14.1.

Plasticity

The fines of granular subbase materials should have a limited plasticity de-

Table 14.1 Recommended particle size distribution for granular subbase materials (TRL Overseas Road Note 31, ref. 2).

Sieve size (mm)	*Passing the sieve (% by mass)*
50	100
37.5	80–100
20	60–100
5	30–100
1.18	17–75
0.3	9–50
0.075	5–25

pending on the moisture regime where the material is used. The liquid limit (LL) and the plasticity index (PI) should not exceed the following values:

- in wet, tropical climates, 35, respectively 6%;
- in seasonally wet climates, 45, respectively 12%;
- in arid and semi-arid climates, 55, respectively 20%.

If the material is too plastic, it may be modified by mixing it with a little cement or hydrated lime.

A minimum CBR of 30% is recommended for subbase materials compacted to the required field density, normally 95% of the maximum dry density using modified Proctor compaction. Given good drainage and a deep-lying water table, the samples should be tested at the optimum water content. If saturation of the subbase is likely to occur, the samples should be tested after four days of soaking. For coarse gradings it may be relevant to replace the Proctor test with vibro compaction and omit the CBR test.

Filter criteria

If the subbase has to protect a drainage layer from blockage by finer materials or prevent mixing of two layers, then additional filter criteria must be met.

14.3.3 Base

Unbound bases may be constructed from a range of different granular materials. In the tropics the most widespread types are:

- natural gravel;
- crushed gravel and crushed rock;
- dry-vibrated macadam.

Natural gravel base

Grading requirements

Table 14.2 shows the recommended particle size distributions for natural gravel suitable for base construction. The 10 mm nominal size should only be used on roads which carry light traffic.

Table 14.2 Recommended particle size distribution for natural gravel base materials (TRL Overseas Road Note 31, ref. 2).

	Passing the sieve (% by mass)		
	Nominal maximum particle size (mm)		
Sieve size (mm)	*37.5*	*20*	*10*
50.0	100	–	–
37.5	80–100	100	–
20.0	60–80	80–100	100
10.0	45–65	55–80	80–100
5.0	30–50	40–60	50–70
2.36	20–40	30–50	35–50
0.425	10–25	12–27	12–30
0.075	5–15	5–15	5–15

To meet the grading requirement, screening and crushing of larger particle sizes may be required. All gradation analyses should be made on materials that have been compacted in order to simulate breakdown of weak particles under construction. For materials whose stability decreases with breakdown, it may be necessary to specify a maximum Los Angeles abrasion value or maximum aggregate impact value.

Plasticity

The fines of the gravel should preferably be non-plastic. The plasticity index (PI) should not exceed 6%. In arid and semi-arid areas the maximum allowable PI can be increased to 12%. In cases where it is difficult to meet the plasticity requirement, the gravel may be modified by mixing with a few per cent of cement or hydrated lime.

CBR

A minimum CBR of 80% is recommended for base materials after being compacted to specified field density. This is normally 98% of the maximum dry density achieved in the modified Proctor compaction test. All samples should be tested after four days of soaking. In arid and semi-arid areas the specified minimum soaked CBR is reduced to 60%. Coarse gradings may be unsuitable for Proctor and CBR tests.

Marginal materials

In many countries natural gravels which do not meet the normal requirements are used successfully for base construction. They include lateritic gravels with gap-graded particle size distribution, calcareous gravels with high plasticity and volcanic gravels with low particle strength. The use of these materials should be encouraged if local experience has proved it viable. However, the use of 'marginal' materials should normally be confined to roads carrying light traffic.

Weathered rock

Granular materials derived from weathering of basic, igneous rocks such as basalt and dolerites may release detrimental plastic fines during construction and future service (Chapter 13). No single test method is able to consistently identify unsuitable materials in this group, and expert advice should be sought when considering their use.

Crushed gravel and crushed rock base

Grading requirement

Crushed gravel is produced by crushing and screening natural gravel and boulders. A proportion of the fine fraction may be natural, uncrushed material. Crushed rock is manufactured by crushing fresh, quarried rock. Recommended grading limits are shown in Table 14.3. The particles should be angular in shape with a flakiness index less than 35%. Specific limits on the maximum Los Angeles abrasion value or the aggregate impact value may be used to ensure adequate durability.

Plasticity

The PI should not exceed 6%. It may be necessary to add a little cement or hydrated lime to meet this requirement. Great care should be taken when dealing with materials originating from weathering of basic igneous rocks.

The compaction requirement is usually the same as for a base course made from natural gravel, i.e. 98% of the maximum dry density obtained in the modified Proctor test.

Dry-vibrated macadam base

Grading requirements

In the UK this material is called dry-bound macadam which is a little misleading as no binder is added to the material. Dry-vibrated macadam consists of single-

Table 14.3 Recommended particle size distribution for crushed gravel and crushed rock base materials (TRL Overseas Road Note 31, ref. 2).

	Passing the sieve (% by mass)		
	Nominal maximum particle size (mm)		
Sieve size (mm)	*37.5*	*28*	*20*
50.0	100	–	–
37.5	95–100	100	–
28.0	–	–	100
20.0	60–80	70–85	90–100
10.0	40–60	50–65	60–75
5.0	25–40	35–55	40–60
2.36	15–30	25–40	30–45
0.425	7–19	12–24	13–27
0.075	5–12	5–12	5–12

sized crushed stones and fine gravel. The recommended nominal size of the stones is 37.5 or 50.0 mm. The fine gravel should be well-graded material passing the 5.0 mm sieve.

Construction

Construction of dry-vibrated macadam involves spreading of stones in a layer of compacted thickness of not more than twice the stone size. The layer is compacted with a steel-wheeled roller. Then dry, fine gravel is spread on to the surface and vibrated down into the cavities between the stones. Any surplus material is brushed off and final compaction carried out. This sequence is repeated until the design thickness is achieved. The method is particularly suitable in dry regions if stone material is readily available and water is scarce.

14.4 STABILIZED PAVEMENT LAYERS

Definition

Stabilization may be defined as any process by which a soil material is improved and made more stable. The most widely used method of stabilization is 'mechanical stabilization' where the material is made more stable by adjusting the particle size distribution, cf. the preceding section on unbound pavement layers. The properties of a soil material may also be improved by addition of a binder, usually cement or hydrated lime. Ref. (5) gives a detailed account of soil stabilization with cement and lime. Recommendations for the criteria that should be met by materials used in the tropics are published in ref. (2).

14.4.1 Cement stabilization

Cement

Portland cement is produced by heating to high temperature a mixture of calcareous materials (such as limestone) and materials containing silicates and aluminates (such as clay or shale). In the presence of water Portland cement forms hydrated calcium silicates and aluminates which in time form a hard matrix in

which the particles with which the cement has been mixed are embedded. Cement is produced all over the world including most developing countries.

Soil materials

In theory most soil materials, with the exception of organic soils and soils containing sulphates, can be stabilized with cement. However, it is difficult to mix cement intimately with clayey materials, and soils with high plasticity are usually best treated with lime. Uniformly graded materials are normally too costly to stabilize with cement as they require a large cement content and are difficult to compact. Also very silty materials may prove difficult because they are sensitive to small changes in water content. It is desirable that materials to be stabilized with cement have a plasticity index (PI) of less than 20% and a coefficient of uniformity of at least 5.

Cement content

The quality of cement stabilized materials is usually assessed on the basis of unconfined strength tests made on compacted samples that have been allowed to harden during a specified time period. Normally the strength increases linearly with the cement content, but at different rates for different soils. An exception is uniformly graded materials where some of the cement acts as a filler which fills the voids between the particles.

Cement modification

Cement is sometimes used only to reduce the plasticity of unbound base and subbase materials. This treatment is called cement modification.

Density

The density is almost as important as the cement content. On average, a 1% increase in density leads to a 10% increase in strength. The cement stabilized material should be compacted as soon as possible following mixing because the cement begins to hydrate as soon as it gets in contact with water. If the compaction is delayed, some of the cemented bonds that have been formed will be broken down and lost.

Water

The presence of sufficient water in a fresh mixture is important both for the compaction and for the hydration reactions to proceed. The hydration is usually only affected at water contents which are too low to allow adequate compaction. Therefore, the water content should be chosen as close as possible to the optimum for the compaction equipment employed. If the optimum water content cannot be obtained, it is preferable to be on the wet side of the optimum rather than on the dry side. The stabilized layer should be kept continuously wet during the hardening process.

Temperature

The relation between strength and temperature for cement stabilized materials is approximately linear except for materials containing pozzolanic materials. Pozzolanic reactions are discussed later.

Cracks

Due to shrinkage and changes in temperature and water content, stabilized layers will crack. The crack pattern is dependent on the strength of the material. Low-strength materials will have frequent but narrow cracks. Cracks in capping layers and subbase layers are not likely to cause any problems as these layers are covered by a thick base. Cracks in a base course, however, may be reflected through the wearing course. Cracks in the wearing course may allow penetration of water leading to deterioration of the pavement. In order to minimize the risk of reflection cracking, the strength of cement stabilized materials is usually not allowed to exceed an upper limit. However, the most effective method to prevent cracking in base courses from reflecting through an asphalt wearing course is to cover the stabilized layer with a layer of unbound, granular material.

Mix design

The following procedure is recommended for selecting the cement content.

First the optimum water content and the maximum dry density should be determined for the soil mixed with different amounts of cement. After mixing soil, cement and water, the samples should be left two hours before being compacted using modified Proctor compaction.

When the compaction tests have been completed, samples for strength tests should be mixed at the optimum water contents obtained. Again the mixtures should be left two hours before being compacted into cubes or test cylinders at 97% of the maximum dry density. The compacted samples should be cured in moist conditions for seven days and soaked in water for seven days. Finally, the samples should be crushed and the cement content needed to achieve the required strength estimated. Recommended strength requirements for different types of stabilized layers are listed in Table 14.4. As laboratory mixing is normally more efficient than field mixing, the cement content for field use is usually chosen 1 or 2% higher than that determined from laboratory tests.

Table 14.4 Recommended strength requirements for cement and lime stabilized subbase and base materials (TRL Road Note ref. 2).

Type	*Unconfined compressive cube strength (MPa)*
Stabilized base (CB1)	3.0–6.0
Stabilized base (CB2)	1.5–3.0
Stabilized subbase	0.75–1.5

14.4.2 Lime stabilization

Lime

Natural limestone ($Ca(CO)_3$) is used to adjust the pH of agricultural soils but has no use as a stabilizing agent in road pavements. The term lime is used here to mean quicklime (calcium oxide, CaO) and hydrated lime (calcium hydroxide, $Ca(OH)_2$). Quicklime is produced by heating limestone in a kiln until carbon dioxide is driven out. Quicklime is usually 'slaked' with water, forming hydrated lime as a fine, dry powder. It is not necessary to use high-quality lime for soil stabilization. In many countries local, low-grade limestone and temporary burning kilns are applied in the production of lime for road construction.

Clay minerals

Lime is effective as a stabilizing agent in most clayey soils which are difficult to stabilize with cement. The stabilizing effect is due to the clay particles. Most clay particles are flat or elongated and thus have a large surface in relation to the unit weight. The clay particles have an ability to adsorb cations on to their surface. Moreover, most clay minerals are pozzolanic, i.e. they will, in the presence of water, react chemically with lime to form cementing products.

Reactions

Lime has an almost instantaneous effect on the plasticity of clayey soils. Even a small addition of lime will increase the plastic limit (PL), as shown in Figure 14.8, while the effect on the liquid limit (LL) is less pronounced. The overall result is a reduction in the plasticity index (PI) and a more friable soil structure, which makes it easier to work and compact the soil.

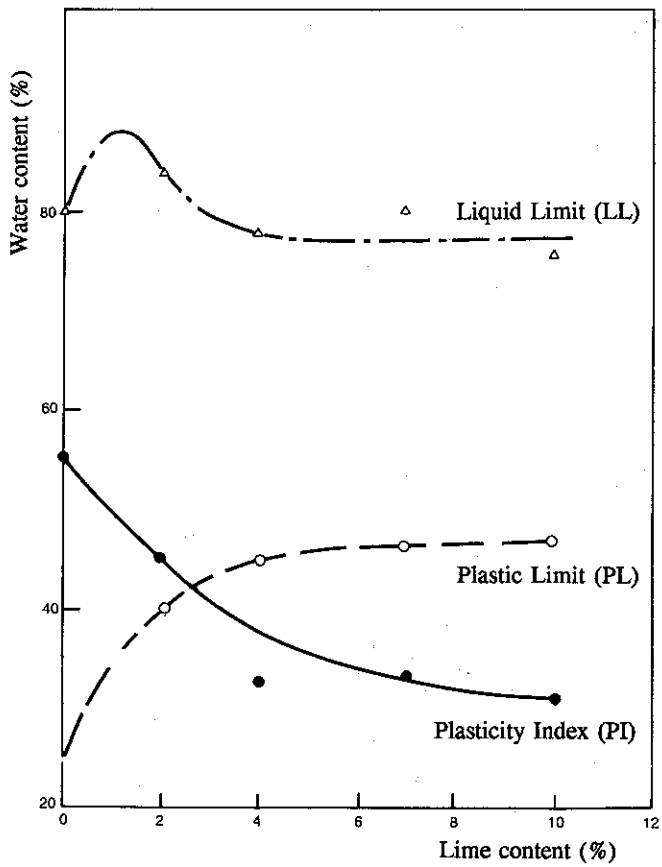


Figure 14.8 Effect of the addition of lime on a London clay (ref. 5). (Crown copyright; reproduced with the permission of the Controller of HMSO.)

The effect of lime on the plasticity is caused by the flocculation of the clay particles brought about by cation exchange in which cations such as sodium and hydrogen are replaced by calcium ions. The decrease in plasticity is followed by an increase in the strength of the soil as measured by the CBR.

If more lime has been added than needed for the cation exchange reactions, the lime will react chemically with the silicates and aluminates of most clay minerals and form cementing products similar to those in Portland cement. In the presence of water the cementing products will hydrate and in time form a hard matrix similar to the matrix in cement stabilized materials.

Lime modification

Lime is often used only as a construction expedient, that is, to improve the workability of wet clayey soils without regard to any time dependent increase in strength. Lime may also be used to reduce the plasticity of unbound base and subbase materials. These treatments are usually called lime modification.

Lime type

The type of lime most widely used for soil stabilization is hydrated lime. Quicklime has a higher drying effect on wet soils, but quicklime is very caustic and creates the risk of skin and eye burns to site personnel. Even hydrated lime should

be handled with care. Where the soil is dry and it is necessary to add water, the lime may be sprayed as a slurry in water, which reduces the dust problem.

Lime content

The strength of soil–lime mixtures increases with increasing lime content, but only up to a certain level. Nothing is gained by adding more lime than that corresponding to the content of reactive clay minerals in the soil.

Density

A high density is equally important for lime stabilized and cement stabilized soils. However, because the pozzolanic reactions in soil–lime mixtures proceed much slower than the hydration of cement, any delay between mixing and compaction is far less critical for lime stabilized soils than for cement treated materials. Actually, after mixing soil and lime, it may be advantageous to leave the mixture for some hours to allow the lime to produce maximum effect on the plasticity. Then the material is remixed and compacted.

Water

Both lime and cement stabilized materials should be compacted at optimum water content. The presence of sufficient water is essential both for compaction and for the pozzolanic reactions and the hydration to proceed.

Temperature

At temperatures below 20–25 °C the pozzolanic reactions take place very slowly. Above 25–30 °C there is a large increase in strength for each degree C rise in temperature. The rapid gain in strength with increasing temperature is the main reason why lime finds more use than does cement for soil stabilization in countries having a warm climate.

Cracks

Lime stabilized materials are subject to cracking for the same reasons as cement stabilized materials but the effects are less pronounced. Continuing pozzolanic reactions may result in 'self-healing' of the cracks.

Carbonation

If air has access to lime stabilized soils during the hardening process, the hydrated lime will react with carbon dioxide from the air and revert to calcium carbonate. The main distress caused by carbonation is loosening of the surface. To prevent carbonation, the stabilized layer should not be allowed to dry out during the hardening process, and the surface should be sealed as soon as possible.

Mix design

The procedure for selecting the lime content depends on the usage of the lime treated soil. Lime modified soils intended for use in capping layers should have a recommended minimum CBR of 15%. Lime stabilized soils in subbase and base courses should satisfy the same strength requirements as cement stabilized soils (Table 14.3). However, the curing period for lime stabilized test cubes or cylinders should be 21 days of moist curing followed by seven days of soaking in contrast to the seven plus seven days of curing used for cement stabilized samples.

Pozzolans

Besides clay minerals, several 'artificial' materials have pozzolanic properties. The most important of these materials is pulverized fuel ash or fly ash, which is a by-product from power stations burning coal-dust. Mixtures of lime and fly ash may be used for stabilizing non-clayey, granular materials. Another use of fly ash is as an additive to Portland cement. When Portland cement reacts with water, hydrated lime is produced in addition to the strength-giving hydrated calcium silicates and aluminates. If fly ash is present, it will react with this lime to produce further cementing materials. 'Fly ash cement' may contain up to 30% fly ash.

The ash produced from burning of agricultural wastes may be rich in reactive silica. Rice husk ash, which is produced in great quantities throughout Asia, has proved to be an excellent pozzolan. The ash from the burning of bagasse (crushed sugar cane) is also known to have pozzolanic properties.

14.4.3 Construction of stabilized layers

Stabilized layers may be constructed using either the mix-in-place method or the plant-mix method. The mix-in-place operation takes place on the construction site where the stabilizer is mixed with the *in situ* subgrade soil or with borrow material placed on the formation of the road. In plant-mix operation the stabilizer and the soil are mixed at a stationary mixing plant.

Mix-in-place

In the mix-in-place method the material to be stabilized is first shaped to the required level and crown. Then an initial scarification or pulverization is performed to specified depth and width. The same type of equipment is used as applied later for mixing stabilizer and soil. After the soil has been loosened, the stabilizer is spread either by mechanical spreaders or by hand. Mechanical spreaders give the most uniform distribution. Attention should be paid to the dust problem. When spreading stabilizer by hand the labourers should be provided with protective clothing and breathing masks.

After spreading, the stabilizer is mixed with the soil using special travelling mixers (rotavators). Motor graders are sometimes applied, but they are inefficient for pulverizing clayey soils and too slow for processing cement stabilized soils. Agricultural disc ploughs or harrows may be used for lime modification of overly wet soils. Also hand mixing has been used, but this highly inefficient method should be discouraged.

If it is necessary to add water to obtain the required optimum moisture content, this may be done through a sprayer in the mixer. In case the mixer is not equipped with a spray system, water should be added and mixed with the soil prior to spreading of the stabilizer. When stabilizing dry soils with lime, the lime is most conveniently distributed as a slurry in water.

Plant-mix

In the plant construction process, mixing is carried out in a central plant and the mixture hauled to the job site in trucks. Clayey materials can usually not be mixed using this process, and plant-mixing is to all intents and purposes only used for cement stabilization of gravelly materials. Plant-mixing provides the best opportunity for obtaining good control on proportioning the materials but the output is lower than in the mix-in-place process. On the construction site the mixture is spread by an asphalt spreader or a spreader box. If graders are used, it is difficult to obtain the right levels and thickness of the stabilized layer, and many of the advantages of plant-mixing are lost.

Compaction

The stabilized layer must be compacted as soon as possible after mixing. Most specifications require compaction of cement stabilized soils to be completed within 2 hours. Delays in lime stabilization are less critical but the compaction should still be finished quickly, particularly in hot climates where evaporation of water and carbonation of lime may create problems. Only when lime is used for modification of wet soils, may the compaction be delayed in order to provide a more workable soil.

Curing

The newly finished layer must be properly cured, i.e. loss of water must be prevented for the first seven days or more. Curing is necessary to (i) ensure continuous hydration of the stabilizer, (ii) reduce shrinkage and (iii) reduce the risk of carbonation of lime. Good curing is particularly important, but also very difficult in hot climates. Frequent spraying of water is commonly used, but this

method is likely to leach stabilizer from the top of the layer. Also, if the spraying is interrupted and the surface allowed to dry, even for a short while, the curing will be ineffective. Spraying of water can be more effective if a layer of sand is first placed on the surface of the stabilized layer. The sand is removed when the curing is complete. The most effective curing method, however, is to apply a very light spray of water followed by a viscous cutback bitumen or a slow-setting emulsion. This will serve as a membrane preventing evaporation of water from the stabilized layer.

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15 Asphalt pavements

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Definition

Asphalt pavements or bituminous pavements consist of selected mineral aggregates bound together by a bituminous binder. Asphalt includes a multitude of different pavement types ranging from thin surface dressings to thick layers of asphalt concrete. This chapter deals with bituminous binders and the most widespread types of asphalt pavement.

15.1 BITUMINOUS BINDERS

Definition

Bitumen (called asphalt cement in the US) is a black to dark brown sticky material composed principally of high-molecular-weight hydrocarbons. In some countries it can be found as a component of natural, rock asphalt, but most bitumen is derived from the distillation of crude oil. Not all crude oils contain bitumen. Crude oils from Indonesia and Nigeria are examples of so-called light crudes containing very little bitumen. Heavy crude oils from the Middle East and South America usually have a large content of bitumen (ref. 1).

Bitumen is a thermoplastic material that gradually liquefies when heated. Bitumen is characterized by its consistency at certain temperatures. The consistency is measured by a penetration test or a viscosity test.

Penetration

The penetration is determined using an empirical test method. A sample of the bitumen is heated to a standard temperature (normally 25 °C) in a small container. A prescribed needle weighing 100 g is allowed to bear on the surface of the bitumen for five seconds. The penetration is defined as the distance, in units of 0.1 mm, that the needle penetrates into the bitumen. The test is illustrated in Figure 15.1.

Viscosity

The viscosity is measured at 60 °C using a capillary viscometer. As bitumen is too viscous to flow readily at 60 °C, a prescribed vacuum is applied to the viscometer to induce flow. The time in seconds required for the bitumen to flow between two timing marks is measured. The recorded time multiplied by a calibration factor for the particular viscometer gives the viscosity in poise.

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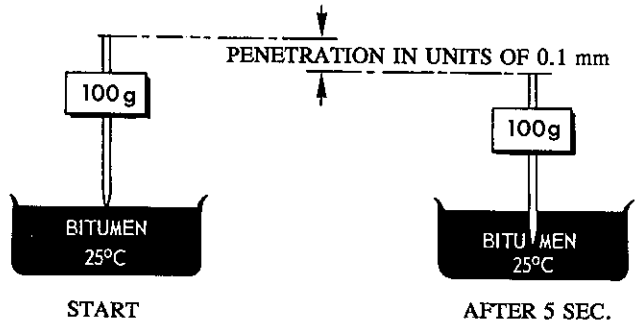


Figure 15.1 Penetration test.

Pen grades

Bitumen is commercially available in several standard grades. For many years the grades have been based on the penetration. British Standard specifies 10 different grades ranging from pen 15 to pen 450. The American standards previously specified five types with pen 40–50 as the hardest and pen 200–300 as the softest.

Viscosity grades

The modern method of grading bitumen is according to its viscosity. As the penetration test is empirical and the viscosity test is scientific, there is no direct relationship between the two. AASHTO has two series of viscosity grades. One is denoted AC (asphalt cement) followed by a number indicating the viscosity in hundreds of poises at 60 °C. The second series is denoted AR followed by a number indicating the viscosity in poises (not hundreds of poises) at 60 °C after the bitumen has been aged. AR stands for 'aged residue'. The ageing is obtained by exposing the bitumen to a jet of hot air for a prescribed period of time. The procedure is intended to subject the sample to hardening conditions approximating those that occur in hot-mix operations.

Other tests

In addition to the penetration test or the viscosity test for defining the consistency, several other tests are required to determine and specify the properties of bitumen. For a description of these tests reference is made to BS or AASHTO.

Liquefied bitumen

Bitumen must be liquefied before it can be sprayed on a surface or mixed with aggregate. Heating is one way of making the bitumen sufficiently fluid. Other ways are to dissolve the bitumen in a light oil or to disperse the bitumen in water as an emulsion. The two products are known as cutback bitumen and emulsified bitumen, respectively. When cutback bitumen is used for pavement construction, the solvent will evaporate on completion of the work leaving the bitumen to perform its function as a binder. With emulsified bitumen the water will evaporate.

15.1.1 Cutback bitumen

Cutback bitumen is produced with different rates of curing (hardening) and different degrees of viscosity. The rate of curing is controlled by the volatility of the solvent used. The viscosity depends principally on the proportion of solvent to bitumen. The more viscous grades require a small amount of heating to make them fluid enough for use.

AASHTO groups

AASHTO specifies three groups of cutback bitumen: rapid curing (RC) where gasoline is used as a solvent, medium curing (MC) containing kerosene and slow

curing (SC) made with diesel oil. The viscosity is measured with a capillary tube viscometer at 60 °C. The flow through the viscometer is induced by gravity: a vacuum is not needed. The viscosity is expressed in centistokes whereas for refined bitumen the viscosity was expressed in poise. (The units of poise and stokes are related to each other by the density of the tested material.)

BS groups

BS only offers specifications for three viscosity grades of cutback bitumen intended for use in surface dressings. The viscosity is measured with a so-called discharge viscometer. By use of this type of viscometer, the viscosity is defined as the time in seconds for 50 ml of the binder to flow through a standard orifice at 40 °C.

15.1.2 Emulsified bitumen

Emulsification process

In the emulsification process, warm bitumen is divided into minute globules and dispersed in water. The machine used in this process is called a colloid mill. In order to discourage coagulation of the bitumen, an emulsifying agent is added to the water. The emulsifying agent provides the bitumen globules with electrical surface charges which prevent the small particles from coalescing. If the charges are negative the emulsion is called anionic. If the charges are positive, the emulsion is called cationic.

Setting of emulsion

When emulsified bitumen is sprayed on to a road surface or mixed with aggregate, the bitumen-phase separates from the water by evaporation of the water and/or neutralization of the electric charges. The manner and rate at which the emulsion breaks or sets largely depend on the properties of the emulsifying agent and the relative proportion of bitumen and water. Both anionic and cationic emulsions are manufactured in several grades. BS distinguishes between rapid, medium and slow breaking types of emulsified bitumen. AASHTO uses the terms rapid, medium and slow setting.

15.1.3 Modifying additives

During the last decade different additives that may improve the properties of bitumen have been developed. Most additives are based on rubber or polymers. By using the right type of modifier the stiffness and the resistance to deformation of asphalt pavements can be increased. These effects are important, particularly for heavily loaded pavements in hot climates.

15.2 SURFACE DRESSING

Definition

A surface dressing (surface treatment in the US) is a wearing course made by a thin film of binder which is sprayed on to the road surface and immediately covered with a single layer of stone chippings of uniform size. A single surface dressing consists of one application of binder and chippings. If two applications are used, the wearing course is called a double surface dressing.

Use

A surface dressing is a simple and inexpensive wearing course. A single surface dressing is adequate as a wearing course on lightly trafficked roads and as a maintenance measure on existing asphalt pavements. Double surface dressings

should be used on new roads expected to carry more than 100 vehicles per day and in cases where the chippings available are particularly poorly shaped or very weak. A surface dressing provides a dust-free and durable running surface with good skid resistance. It has no structural strength itself, but it provides a waterproof seal and prevents the ingress of surface water thus preserving the inherent strength of the pavement and subgrade.

Reference

A guide to surface dressing in tropical and subtropical countries has been published in Overseas Road Note 3 by the Transport and Road Research Laboratory in the UK (ref. 2). The main points from Overseas Road Note 3 are presented in the following.

15.2.1 Selection of chippings

Strength

Ideally, the chippings should be single-sized, cubical, strong, durable, clean and dust-free. Specifications for maximum aggregate crushing value (ACV) typically lie in the range of 20–35. In wet climates the chippings should be resistant to polishing under the action of traffic. British specifications call for a minimum polished stone value (PSV) of 45–60 depending on traffic and site characteristics. In practice the chippings available often fall short of the ideal.

Size

The action of the traffic on a surface dressing gradually forces the chippings down into the underlying surface. This embedment process occurs more rapidly when the underlying road surface is soft rather than hard, and when the volume of commercial traffic is high. Consequently, large chippings are required on soft surfaces or where traffic is heavy, and small chippings should be used for hard surfaces and light traffic. The nominal size of chippings used for surface dressings is usually 6, 10, 14, or 20 mm. The appropriate size is selected from Table 15.1.

For double surface dressings the size of chippings for the first layer should also be selected according to Table 15.1. The size of the chippings for the second layer should be about half the size of the first layer to promote good interlock between layers. Sand may sometimes be used as an alternative to chippings for the second layer.

There is one exception to the rule that larger chippings should be used in the first dressing. If the existing surface is very hard, such as a newly constructed cement stabilized base course, a 'pad coat' of 6 mm chippings should be applied first followed by a second layer with 10 or 14 mm chippings.

Table 15.1 Recommended nominal chipping size in mm for surface dressing (TRRL Overseas Road Note 3, ref. 2).

No. of commercial vehicles per day	*Surface category*			
	Very hard	*Hard*	*Normal*	*Soft*
<20	6	6	6	10
20–200	6	6	10	14
200–1000	6	10	14	14
1000–2000	10	14	14	20
>2000	10	14	20	*

*Not suitable for surface dressing.

15.2.2 Selection of binder

Both refined bitumen, cutback bitumen and emulsified bitumen may be used as a binder for surface dressings. The use of cutbacks is decreasing because the solvents are high-energy products that are lost by evaporation and because of environmental problems. Emulsions have a great potential, especially in developing countries, because they can be manufactured locally using small mobile plants. Also, emulsions can be made with a harder basic bitumen than cutbacks, and a hard bitumen is advantageous in hot, tropical climates. The use of emulsified bitumen for surface dressings is increasing rapidly in Asia whereas cutbacks are still commonly used in most African countries.

The correct choice of binder is critical. The binder must be sufficiently fluid at road temperature to wet the road surface and the chippings. At the same time the binder must be sufficiently viscous not to drain off from the road surface and strong enough to retain the chippings when the road is opened to traffic. In the tropics the road temperature typically lies between 25 °C and 50 °C.

Bitumen and cutback

Figure 15.2 shows the permissible range of binder viscosity at various road temperatures and the appropriate grades of bitumen and cutback.

Emulsion

Figure 15.2 does not apply to emulsified bitumen. Emulsified bitumen for surface dressing should be rapid or medium curing. Anionic emulsion should be chosen when the chippings are made from calcareous rocks and cationic emulsion when the rock is igneous. Most emulsified bitumens have a low viscosity and easily wet the road surface and chippings. Because of the low viscosity, emulsion may not be applicable if a high rate of spray is required, as is the case for large-sized chippings. The light-fluid emulsion may drain off from the high parts of the road surface before breaking.

15.2.3 Rate of spread of binder and chippings

ALD

Once the size of chippings and the type of the binder have been selected, the next step is to determine the required rate of spread of the binder. The rate of spread depends on the average thickness of the stone chippings when they have settled in their final position on the road. It is assumed that the particles will settle with their least dimension being vertical. The average least dimension (ALD) of the selected chippings may be measured manually on a representative sample.

Binder

The rate of application of binder also depends on the type of chippings, the level of traffic, the condition of the existing road surface, and the climate. An appropriate factor for each of the four sets of conditions is selected from Table 15.2 and added together to give a total weighting factor. The ALD and the weighting factor are then used with Figure 15.3 to obtain the required rate of application of binder.

Chippings

Chippings should be spread at a rate corresponding to a single, tightly packed layer plus a 10% allowance to ensure complete coverage. A rough guide is given at the top of Figure 15.3.

15.2.4 Construction of surface dressing

Untreated surface

Before applying a surface dressing, the surface must be carefully prepared. Previously untreated surfaces may be 'primed' with light fluid cutback bitumen in order

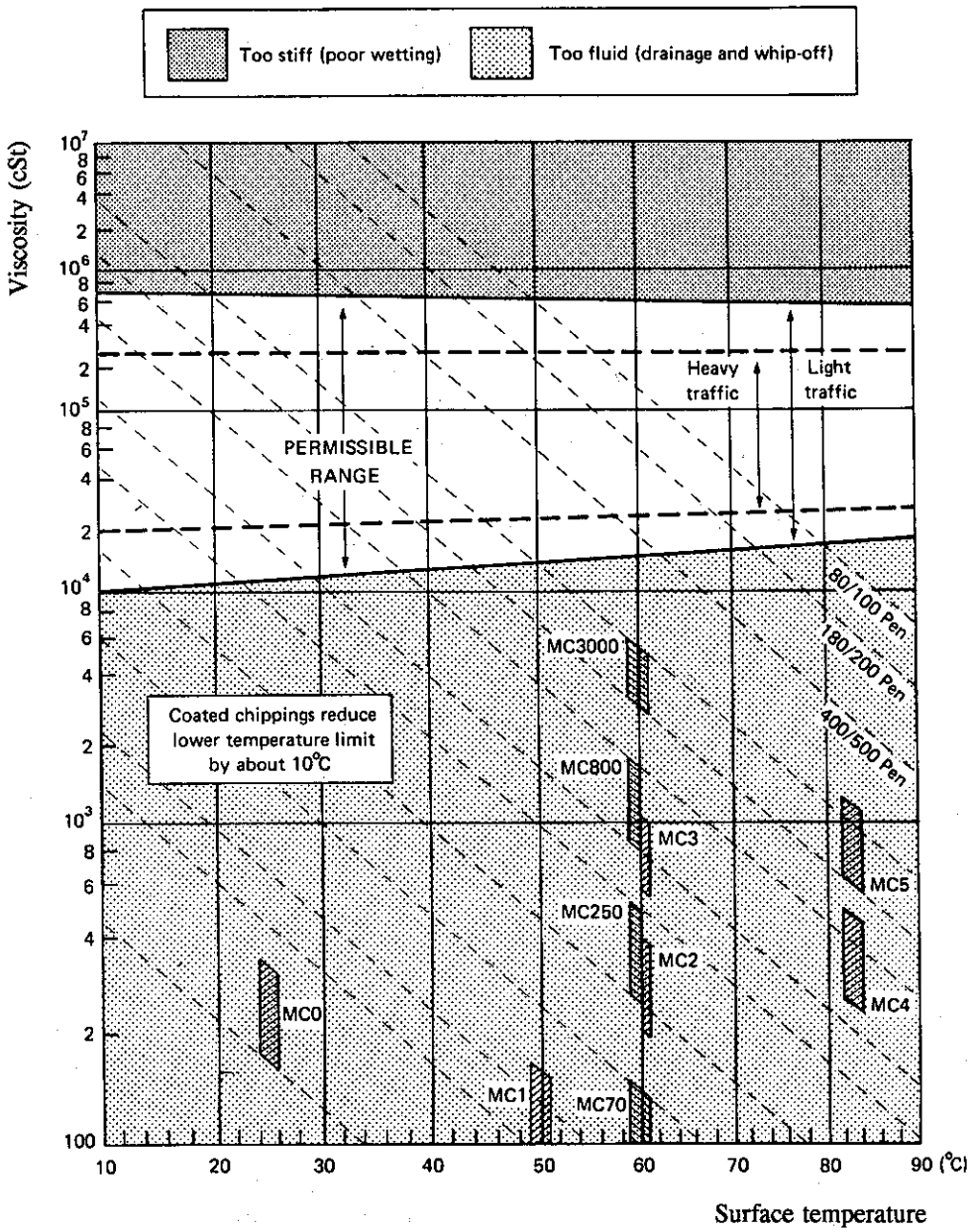


Figure 15.2 Choice of binder for surface dressing (TRRL Overseas Road Note 3, ref. 2).

to strengthen the top of the base and promote adhesion between the base and the surface dressing to be applied.

Old pavement

Old asphalt pavements must be repaired and all potholes patched with asphalt materials producing a surface texture similar to the adjacent pavement. Traffic should be allowed to run over the patched road for some months.

Table 15.2 Condition factors for determining the rate of application of binder for surface dressing (TRRL Overseas Road Note 3, ref. 2).

Vehicles per day		*Existing surface*	
0–50	+3	Untreated base	+6
50–250	+1	Very lean asphalt	+4
250–500	0	Lean asphalt	0
500–1500	−1	Average asphalt	−1
1500–3000	−3	Very rich asphalt	−3
>3000	−5		
Climate		*Type of chippings*	
Wet and cold	+2	Round/dusty	+2
Wet and hot	+1	Cubical	0
Temperate	0	Flaky	−2
Dry and hot	−1	Precoated	−2
Very dry and hot	−2		

Cleaning

Immediately prior to spraying the binder, the road should be swept clean. The surface must be dust-free and dry. Dressing operations should never be started when rain is expected within the first few hours.

Spreading of binder

The binder must be applied uniformly at the correct rate of spread. Watering cans may be used for minor works such as patching. A more controllable method is to use hand lances. With a hand lance it is possible to produce an acceptable, uniform rate of spread, but it is difficult to achieve a specified rate of spread. Successful spreading of binder on large areas requires the use of a mechanical bulk distributor.

Spreading of chippings

Immediately after the binder has been applied, the chippings should be spread. This ensures maximum possible wetting of the chippings. The chippings can be spread by hand with a good result, but a mechanical chip spreader facilitates an even distribution and rapid application.

Rolling

Following the distribution of chippings, the layer should be rolled in order to seat the aggregate properly in the bitumen. Traditionally, steel-wheeled rollers have been used for this purpose, but steel-wheeled rollers tend to crush weak aggregate, and pneumatic-tyred rollers are to be preferred.

Brooming

As it is necessary to apply a slight excess of chippings, some loose material on a new surface dressing will inevitably occur. This excess may be removed by brooming after the traffic has been permitted to run over the road surface for some days.

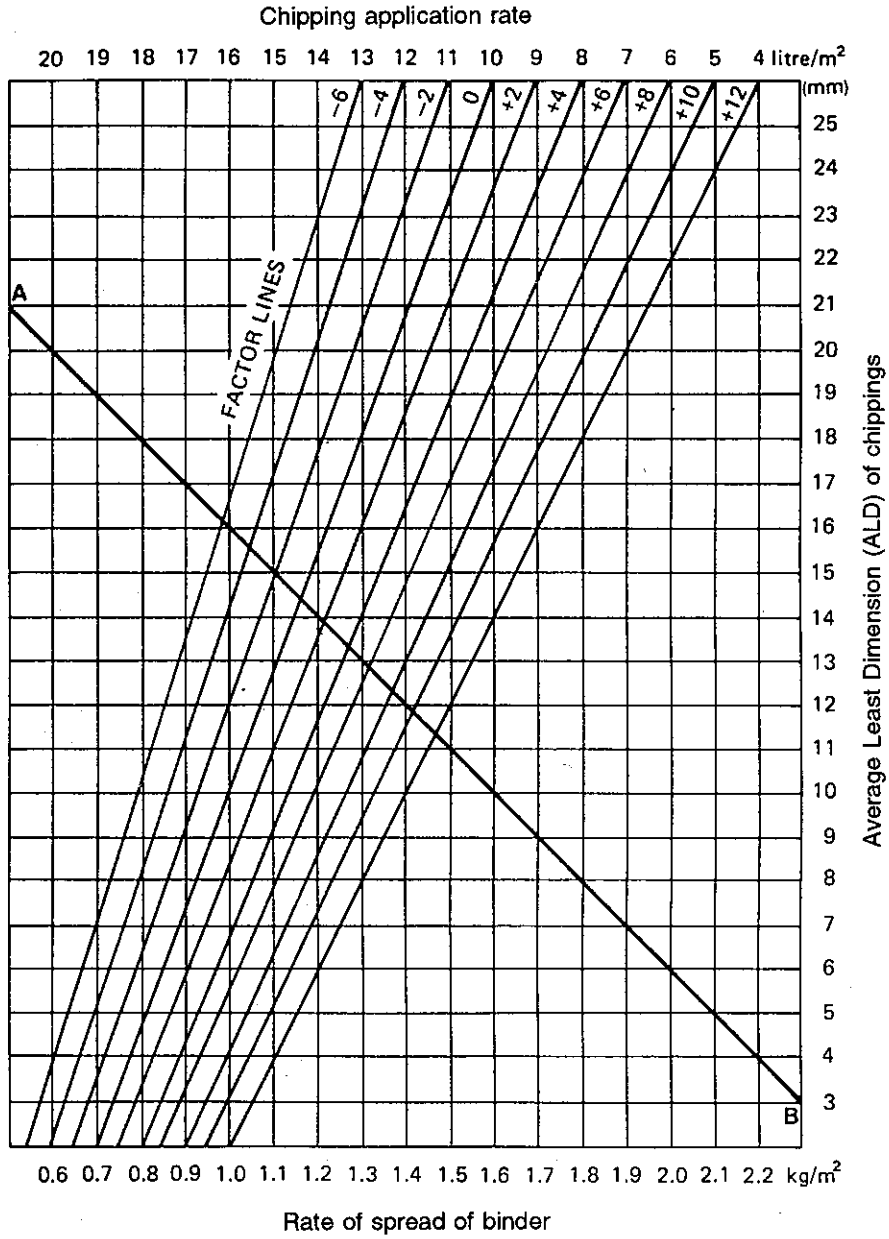
Double surface dressing

Construction procedures for double-surface dressings are essentially the same as those for single-surface dressings except that the first dressing is allowed to cure before the process is repeated and the second dressing is placed.

15.3 PREMIXED ASPHALT

Definition

Premixed asphalt is a paving material manufactured by mixing aggregates, filler and bitumen. Most premixed asphalt is mixed and placed hot, hence the American



Rate of spread of binder. The rate of spread of cutback bitumen is determined by the intercept between the factor line and the appropriate ALD line. For penetration bitumen the rate determined from the chart should be reduced by 10%. For emulsified bitumen the rate should be multiplied by (90/b) where b is the percentage of bitumen in the emulsion. For slow traffic or steep climbing grades the rate should be reduced by 10%. For fast traffic or steep down grades the rate should be increased by 10 to 20%.

Chipping application rate. The rate is determined by the intercept between the factor line and line AB.

Figure 15.3 Rate of spread of binder and chippings (TRRL Overseas Road Note 3, ref. 2).

name 'hot-mix'. Premixed asphalt is used in the construction of wearing courses, binder courses and base courses.

15.3.1 Types of premixed asphalt

Asphalt concrete

Many different types of premixed asphalt are manufactured throughout the world. By far the most common type is asphalt concrete. Asphalt concrete is made from a continuously graded aggregate, and it relies for its strength on the interlock between the aggregate particles and on the properties of the mortar of bitumen, fine aggregate and filler. Asphalt concrete of different qualities may used for the construction of wearing course, binder course and base course.

Rolled asphalt

Rolled asphalt is a type of premixed asphalt developed in the UK, but according to Road Note 31 this asphalt type has also performed well in the tropics. It is a gap-graded mix which relies for its properties mainly on the asphalt mortar. Rolled asphalt can be used for the same pavement layers as asphalt concrete.

Sand asphalt

Sand asphalt – or bitumen stabilized sand – is a mixture of sand and bitumen, cutback bitumen or emulsified bitumen. Sand asphalt is an alternative surfacing material for roads with medium traffic in areas lacking coarse aggregate.

Asphalt macadam

Asphalt macadam is an asphalt construction using coarse, open graded aggregate. The strength is mainly due to the interlock between the aggregate particles. Asphalt macadam is suitable for the construction of the wearing course and the base course. When used as a wearing course, it is usually sealed by sprayed-on-binder blotted with fine aggregate. Asphalt macadam should not be confused with 'bitumen macadam' as defined in Road Note 31. Bitumen macadam is a continuously graded mix similar to asphalt concrete but with a less dense aggregate structure.

Mix design

Premixed asphalt is designed using a special mix design method or it is made to recipe specifications without reference to a formal design method. Asphalt concrete and sand asphalt are usually designed using the Marshall method, whereas rolled asphalt and asphalt macadam are often made to recipe specification. Mix design of asphalt concrete – the most common type of premixed asphalt – is discussed in a later section.

15.3.2 General requirements

General properties and specifications for premixed asphalt suitable for tropical conditions are given in Road Note 31 (ref. 3). Some of the recommendations are outlined in the following.

To facilitate placing and compaction of the asphalt, the fresh mix should have good workability. In order to perform satisfactorily, the finished asphalt layer needs to have:

- high stiffness in order to reduce the stresses transmitted to the underlying layer;
- high resistance to deformation;
- high resistance to fatigue;
- high resistance to weathering (good durability);
- low permeability in order to prevent intrusion of air and water.

Some of these requirements are conflicting. The durability of a mix is improved

by increasing the bitumen content, but more bitumen normally reduces the stiffness and resistance to deformation. The art of asphalt mix design is to produce a mix which possesses an acceptable balance of properties. Mix requirements are more critical in the tropics than in temperate climates because of higher temperatures and often higher axle loads. High temperatures reduce the stiffness of asphalt and also cause the bitumen to oxidize and harden more rapidly. High axle loads increase the risk of secondary compaction and plastic deformation of the asphalt.

Aggregates

Aggregates are usually categorized as coarse aggregate, fine aggregate and filler. Coarse aggregate is defined as mineral aggregate retained on the 2.36 mm sieve, fine aggregate is material passing the 2.36 mm sieve whereas filler is the fines passing the 0.075 mm sieve.

Coarse aggregate

Coarse aggregate for premixed asphalt should be produced by crushing rock or natural gravel to obtain angular, rough-textured particles with good mechanical interlock. The coarse aggregate should be:

- clean;
- low-absorptive;
- angular in shape;
- weather-resistant;
- resistant to abrasion;
- resistant to polishing if used in wearing courses.

Fine aggregate

Fine aggregate can be crushed rock or natural sand. The fine aggregate should be:

- clean
- weather-resistant
- non-plastic.

Table 15.3 shows the most important requirements recommended for coarse and fine aggregate.

Filler

Filler consists of the natural fines inherent in the coarse and fine aggregate. Crushed rock fines, Portland cement or hydrated lime may be added in order to

Table 15.3 Recommended quality requirements for coarse and fine aggregate intended for use in premixed asphalt (TRL Overseas Road Note 31, ref. 3).

Coarse aggregate		*Fine aggregate*	
Passing sieve 0.075 mm	<5%	Passing sieve 0.075 mm:	
Water absorption	<2%	wearing course	<8–17%
Flakiness Index	<45%	other courses	<22%
Soundness:		Sand equivalent:	
in sodium	<12%	wearing course	min. 35–40%
in magnesium	<18%	other courses	min. 45–50%
Los Angeles Abrasion:		Soundness:	
wearing course	<30%	in sodium	<15%
other courses	<35%	in magnesium	<20%
Aggregate Impact Value	<25%		
Polished Stone Value:			
wearing course only	min. 50–75		

obtain the required proportion of filler in the mix. Addition of hydrated lime is particularly useful as it improves the adhesion of the bitumen to the aggregate.

Bitumen

The bitumen used for production of hot premixed asphalt in the tropics is usually pen 40–50, pen 60–70, or pen 80–100. The bitumen content depends on the asphalt type, but normally it lies between 4 and 7% by mass of total asphalt mix.

15.3.3 Asphalt mixing

Hand mixing

In some developing countries asphalt is produced *in situ* by hand mixing. However, high-quality premixed asphalt is produced at stationary or portable mixing plants. Hand mixing is discussed in Chapter 19. Asphalt production at mixing plants is described in the following.

In general, most asphalt mixing plants can be categorized as either a batch facility or a drum-mix facility.

Batch mixer

In a batch facility (Figure 15.4) aggregates in the approximate proportion needed are drawn from the storage (cold bins) on to a conveyer belt which leads to an elevator. The elevator delivers the combined aggregates into a dryer.

Dryer

The dryer is an inclined, rotating, steel drum with a gas or oil heating unit (heating flame) in the lower end. The combined aggregates enter the dryer at the upper end and are moved against the flame by the rotating action of the drum. As the aggregates pass through the dryer, the moisture is driven off, and the material is heated to a temperature somewhat higher than that required for mixing.

Screening unit

The hot, dry aggregates go up the elevator to a screening unit, where they are separated into several sizes. The different fractions are stored temporarily in 'hot bins' placed below the screens. A prescribed amount of aggregates is successively drawn from the hot bins into a weigh box located below the bins. Filler is added from a separate storage bin.

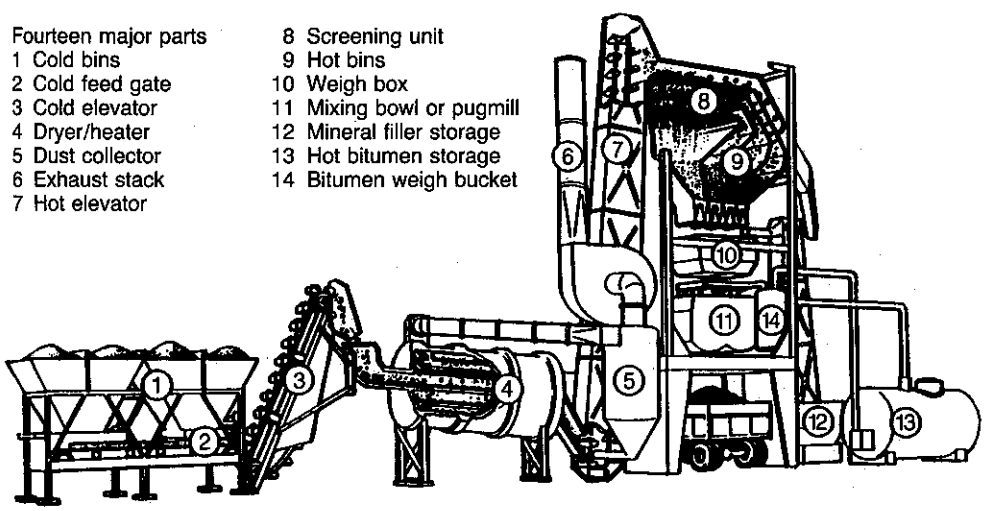


Figure 15.4 Batch mixer (ref. 4).

Mixer

The proportioned aggregates and filler are discharged into a twin shaft mixer (pugmill) where a measured amount of hot bitumen is added through a spray bar and the materials thoroughly blended. The mixed batch of asphalt is discharged into a truck or transferred to a storage bin.

At some batch facilities the heated aggregates are not separated by size into different hot bins. With accurate feeders at the stockpiles, satisfactory grading control can be obtained without this rescreening.

Dust collector

All modern asphalt mixing facilities are supplied with dust collectors in order to abate the dust nuisance that otherwise results from exhaust air from the dryer. The collected dust is usually returned to the hot aggregate as it emerges from the dryer.

Calibration

In order to produce a high-quality asphalt mix it is of paramount importance that all ingredients are measured accurately. Accordingly all weights and metering devices should be calibrated regularly.

Temperature

The temperature of the asphalt when it leaves the mixer should be high enough to get good particle coating, and for laying and compacting. However, to avoid burning and to reduce hardening of the binder, the bitumen should never reach a temperature higher than about 160–175 °C depending on the bitumen grade. Temperature measuring instruments should be placed in the dryer discharge, in the bitumen storage tank and in the bitumen feeder line and the instruments should be checked frequently.

Drum mixer

The principal feature of a drum-mix facility is that the drying drum also serves as a mixer and that the output of asphalt mixture is continuous (Figure 15.5). A drum-mix facility has no hot aggregate screens and hot bins. The cold aggregate storage bins are supplied with accurate metering devices which feed a continuous stream of corrected graded aggregates and filler at a controlled rate into the drum.

Drum

In contrast to a batch facility, the heating flame is placed in the upper end of the drum where the aggregates are introduced, and the material moves through the drum away from the flame. A bitumen metering device feeds hot bitumen into the drum at a controlled rate. The binder is introduced midway in the drum to avoid contact with the open flame. From the drum the hot asphalt mixture is taken by a conveyer to a surge bin from which it is loaded into trucks.

A drum mixer is simpler and cheaper to operate than a batch plant. However, drum mixers (and batch plants without hot screens) require a steady flow of aggregates with consistent moisture contents. Furthermore, it causes considerable disruption when a drum mixer has to change from one asphalt type to another.

15.3.4 Asphalt paving

Surface preparation

The riding quality of the pavement surface depends largely on proper construction and preparation of the foundation.

Unpaved surface

Unpaved surfaces should be shaped and rolled so that the paving equipment has no difficulty in placing the material in a uniform thickness to a smooth grade. Loose granular particles should be swept from the surface of the road, but care should be taken not to dislodge the aggregate in the surface of the road. The clean surface should be primed with cutback bitumen using a pressure distributor. Under average conditions the rate of spray should be from 0.9 to 2.3 litres per m^2.

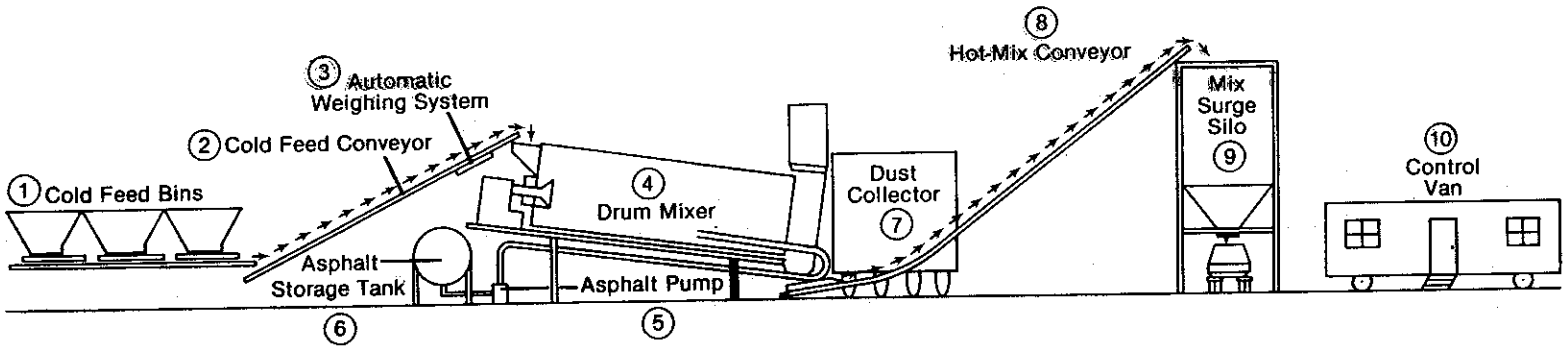


Figure 15.5 Drum mixer (ref. 5).

Emulsified bitumen may be used, but under most circumstances it is more difficult for an emulsion than for a cutback bitumen to penetrate a granular base.

Old paved surface

Old asphalt pavements should be carefully repaired before receiving a new overlay. Potholes should be patched, and the patches should be deep enough to strengthen the base. All bleeding and old unsuitable patches, excess asphalt crack sealer, and loose scales should be removed from the surface of the existing pavement. All depressions of 25 mm or more should be overlaid with a levelling course and compacted. After cleaning with power brooms, a very thin tack coat of emulsified bitumen should be applied in order to ensure a bond between the old and new asphalt.

Spreading asphalt

Most premixed asphalt is placed by asphalt pavers. The asphalt is brought to the paving site in trucks and deposited directly into the paver. The asphalt paver spreads the mixture in a uniform layer of desired thickness and shape, ready for compaction.

Asphalt paver

An asphalt paver essentially consists of a tractor unit and a screed unit. The tractor unit provides the motive power through crawlers or pneumatic tyres travelling on the subbase or base course. A side view of a paver is shown in Figure 15.6. The mix is dumped into the receiving hopper at the front of the machine. Slat feeders carry the mix back through the control gates to the spreading screws, which distribute the mix in front of the screed unit.

Screed unit

The screed unit is attached to the tractor unit by two long arms that pivot well forward on the paver. When the screed is pulled into the material deposited in front of it, it automatically rides up or down seeking the level where the path of its flat bottom surface is parallel to the direction of pull. The thickness of the mat can be increased or decreased by adjusting the angle between the screed plate and the pull arm or by moving the pivot point of the pull arm vertically. The screed may be controlled manually or automatically. An automatic control gets its information from a sensing device riding on a stringline, or from a ski riding on an adjacent lane. The screed unit strikes off, partially compacts, and irons the surface of the mat as it is pulled forward. Most screeds are vibrating but some units have a tamper bar which moves up and down.

Compaction

The purpose of compacting an asphalt pavement is to achieve the optimum air

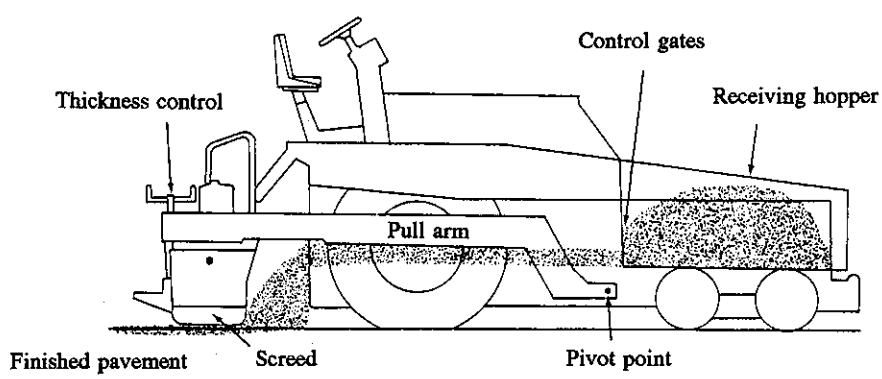


Figure 15.6 Asphalt paver (ref. 5).

void content and provide a smooth riding surface. Compaction is done by any of several types of rollers which, by their weight or by exertion of dynamic force, compact the pavement mat by driving over it in a specific pattern. Self-propelled rollers such as steel-wheeled tandem rollers, vibrating rollers, and pneumatic-tyred rollers are normally required for compaction of high-quality asphalt mixtures.

Number of rollers

Two or more rollers are needed on most projects other than small jobs. Rolling should begin as soon as possible after the hot asphalt has been spread. The rolling operation should start from the edge of spread on the low side of the lane being paved and progress towards the high side. The reason for this is that hot asphalt mixture tends to migrate towards the low side of the spread under the action of a roller. Rollers should move at a slow but uniform speed with the drive wheel facing the paver. The line of rolling and the direction of rolling should be changed slowly in order to avoid displacement of the mix. Roller wheels should be kept moist with only enough water to avoid picking up material. Heavy equipment, including rollers, should not be permitted to stand on the finished surface before it has cooled.

Layer thickness

Premixed asphalt, except sand asphalt, is usually placed in layers with a compacted thickness between two and three times the maximum size of the aggregate.

15.3.5 Design of asphalt concrete

Asphalt

For asphalt concrete surfacings the combined aggregates should be well (continuously) graded. Table 15.4 shows the recommended grading limits.

Bitumen

The binder in asphalt concrete is usually bitumen with a penetration of 60–70 or 80–100. The binder content varies between 5 and 7% by mass of total asphalt mix. The optimum binder content for a particular mixture is normally obtained from a Marshall test as outlined in the next section.

Table 15.4 Recommended particle size distribution for combined aggregate for use in asphalt concrete wearing course and binder course (TRL Overseas Road Note 31, ref. 3).

	Passing the sieve (% by mass)		
	Wearing course		*Binder course*
Sieve size (mm)	*WC1*	*WC2*	*BC1*
28.0	–	–	100
20.0	100	–	80–100
14.0	80–100	100	60–80
5.0	54–72	62–80	36–56
2.36	42–58	44–60	28–44
1.18	34–48	36–50	20–34
0.6	26–38	28–40	15–27
0.3	18–28	20–30	10–20
0.15	12–20	12–20	5–13
0.075	6–12	6–12	2–6

The Marshall method

Test specimens

The Marshall test uses cylindrical test specimens with a diameter of 102 mm and a height of about 64 mm. The specimens are prepared using a standard procedure for heating, mixing and compacting the asphalt mix. Test specimens are made for a range of bitumen contents within the prescribed limits. At least three specimens are provided for each bitumen content to facilitate the provision of adequate data. Each specimen is then subjected to a density test and a stability–flow test.

Density

The density test consists of weighing the specimen in air and in water. The volume is the difference between the mass in air and the mass in water. The bulk density is calculated as the ratio between the mass in air and the volume.

Voids

Two more characteristics are calculated: (i) the air voids in the mix, i.e. the total volume of the small pockets of air between the coated aggregate particles in the compacted mixture, and (ii) the voids in the aggregate, i.e. the total volume of air and bitumen in the compacted mixture. The air voids and the voids in aggregate are calculated from the bulk density of the compacted specimen, the bitumen content, the specific gravity of the bitumen, and the apparent specific gravity of the aggregate.

Stability and flow

In conducting the stability–flow test, the specimen is heated to 60 °C in a water bath. The specimen is then placed in the Marshall testing machine between two collar-like testing heads and compressed radially at a constant rate of strain (Figure 15.7). The Marshall stability is the maximum load resistance in units of Newtons. The flow value is the total deformation of the specimen at the maximum load.

Optimum binder content

When testing is complete, plots are prepared for bitumen content versus (1) density, (2) percentage of air voids in mix, (3) percentages of voids in aggregate, (4) stability, and (5) flow as shown in Figure 15.8.

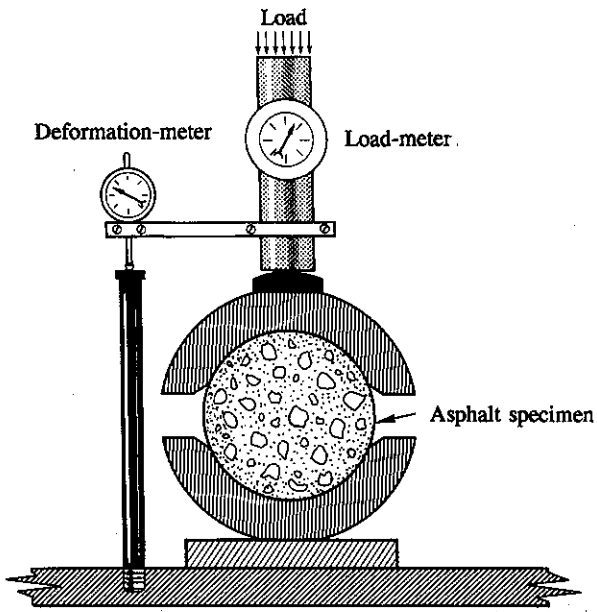


Figure 15.7 Marshall testing machine.

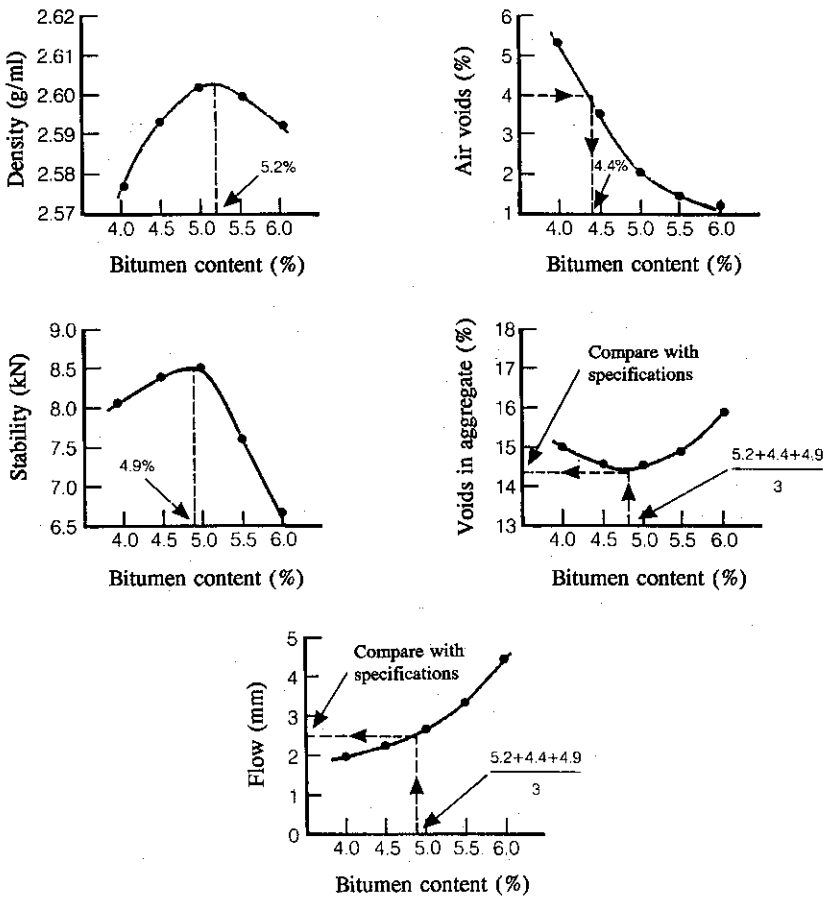


Figure 15.8 Results from a Marshall test (ref. 4).

From the curves are read the bitumen contents corresponding to:

- maximum density;
- median of required limits for percentage of air voids in mix;
- maximum stability.

The optimum bitumen content is calculated as the average of the three bitumen contents.

Marshall criteria

Stability, flow, air voids in mix, and voids in aggregate at the optimum bitumen content are now read from the appropriate graphs and compared to the design criteria. Table 15.5 shows recommended Marshall criteria for asphalt concrete wearing course and binder course. If one or more design criteria are not met, the grading and/or the quality of the aggregate must be adjusted and new Marshall tests made until satisfactory results are achieved.

Severe sites

It has previously been mentioned that the environment for asphalt pavements is often very severe in the tropics. Severe conditions consist of a combination of high temperature, heavy axle loads and slow moving heavy vehicles. Under severe conditions asphalt concrete may experience secondary compaction in the wheel

Table 15.5 Recommended Marshall design criteria for asphalt concrete wearing course and binder course (TRL Overseas Road Note 31, ref. 3).

Design traffic load[1] (10^6 esa)	<1.5	1.5–10.0	>10.0	Severe sites
Compaction level (no. of blows)	2 × 50	2 × 75	2 × 75	2 × 75
Stability min. (kN)	3.5	6.0	7.0	9.0
Flow min. 2 mm				
Air voids 3–5%[2]				
Voids in aggregate: min. 14% for nominal particle size 20 mm min. 18% for nominal particle size 5 mm				

[1] The concept of esa (equivalent standard axles) is explained in Chapter 16.
[2] On severe sites there should still be at least 3% air voids at refusal density.

paths leading to plastic failure. It has been shown that plastic deformation occurs very rapidly once the air voids in the asphalt have fallen below 3%.

For asphalt concrete on severe sites it is recommended that the design binder content, as determined from the Marshall test, is adjusted in order to secure a minimum of air voids of 3% at 'refusal density'. Refusal density is obtained by compacting Marshall specimens until no further increase in density occurs. As seen from Table 15.5 the Marshall specimens are normally compacted using 50 or 75 blows on each face. Up to 500 blows may be necessary to reach refusal density.

Modified bitumen

On severe sites consideration may also be given to using asphalt concrete containing modified bitumen in order to improve the resistance to deformation.

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16 Pavement design

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16.1 INTRODUCTION

The structural design of road pavements differs in some important aspects from other civil engineering design problems, such as design of structures and foundations where most of the common design approaches can be considered as an either/or problem. Either the structure fails immediately or at a later stage, or it lasts almost forever. Pavements normally deteriorate gradually.

Deterioration type

It is necessary to distinguish between functional deterioration and structural deterioration. Functional and structural deterioration refer to the type of deterioration, not to the cause of the deterioration. In the American literature the two concepts are often used to describe the causal relation.

Functional deterioration

Functional deterioration of a pavement means that the riding quality of the road decreases, i.e. the road gives poorer service to the users. Poorer service may mean that the road becomes more uncomfortable to drive on, but it may also mean that safety is reduced.

Structural deterioration

Structural deterioration indicates that the pavement layers lose some of their bearing capacity. It may be due to cracks in bitumen or cement-bound layers or crushing of unbound layers. Structural deterioration often results in functional deterioration, but this is not always the case.

Terminal condition

The perfect pavement design system would specify the magnitude of functional and/or structural deterioration that is allowed before the pavement is deemed as having failed, i.e. the terminal condition. However, only a few design methods define the terminal condition explicitly. The problem is exemplified by the following quotation from ref. (1):

> 'Two designers are given the task of designing a pavement of certain materials for certain traffic and environment for 20 years. The first might consider his

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job to be properly done if not a single crack occurred in 20 years while the second might be satisfied if the last truck that was able to get over the pavement made its trip 20 years from the date of construction.'

It is clear that the two resulting pavement designs would be rather different.

System approach

Most design systems are limited to a calculation of the required thicknesses of given pavement materials to achieve a certain 'life' of the pavement, i.e. prediction of when the pavement has reached the maximum allowable deterioration. If a 'systems approach' is used and the road user costs are included in the total economic evaluation, or if the design is an element of a pavement management system the design method should be capable of predicting the deterioration over time and the consequence of changes to traffic loading, pavement structure and road maintenance.

Deterioration

The need to predict the deterioration over time is illustrated in Figure 16.1 where the conditions of two pavements are plotted as a function of time. The two pavements reach the terminal condition at the same time: pavement 1 after one maintenance step only; pavement 2 after several maintenance jobs. However, during the whole time period, the quality of pavement 1 is better than the quality of pavement 2. The project to be chosen depends on several circumstances, unless the project that is cheapest to construct has to be chosen because of limited funds. The total costs inflicted on the road authority for constructing and maintaining the road should be balanced against the road user costs. It is difficult to evaluate the user costs, and it is difficult to compare the two categories of costs, but this does not justify leaving out the user costs. When the user costs are not calculated, they are normally considered indirectly, i.e. roads with heavy traffic are built to a higher functional standard than roads with light traffic.

The problems mentioned above are not specific to developing countries, but most developing countries lack the tradition of pavement design based on extensive experience as is common in most developed countries. If, for a specific project in a developing country, it is possible to freely choose the design method, the method should be checked or calibrated to the local conditions, but the validation

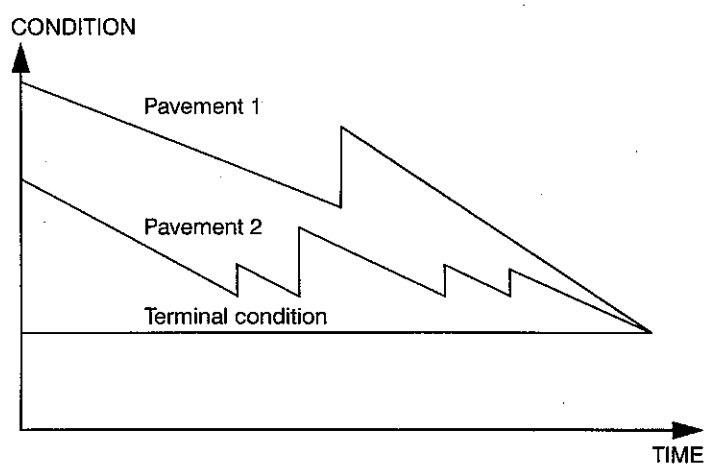


Figure 16.1 Deterioration as function of time.

is difficult if the method describes the deterioration in a way that cannot be quantified.

HDM model

The HDM model (discussed in Chapter 4) predicts the pavement deterioration for a number of different distress modes. Consequently, the HDM model should in principle be well suited for pavement design. However, if the HDM deterioration submodels are used for detailed design of pavements, the results are often not compatible with the designs derived from traditional pavement design methods. The reason is that these models are primarily based on observations of existing roads.

If all the observed pavements are perfectly designed for the traffic they carry, there will be no correlation between the deterioration and the bearing capacity of the pavements. A correlation analysis will show that only age and present condition will be correlated with the deterioration. However, as pavements are not perfectly designed for the traffic they carry, there will be some correlation between deterioration and bearing capacity but the correlation will be weak. This is exactly the case with the HDM deterioration submodels.

For pavements that are in balance, i.e. the pavement structures that correspond reasonably well to the traffic loading, such deterioration models would be quite satisfactory. The future deterioration will primarily depend on the present condition and age. If, however, the traffic loading for some reason changes, or if the pavements are not in balance and need strengthening, these deterioration models are of dubious value. The same applies to the design of new roads.

Two methods developed for pavement design in tropical regions are presented in this chapter: Road Note 31 and the SATCC design guide. The analytical–empirical method is introduced and the design of gravel roads is discussed. The last section deals with the design of strengthening overlays.

16.2 ROAD NOTE 31

16.2.1 Background

Overseas Road Note 31 (ref. 2) is published by the Transport Research Laboratory (TRL) and contains recommendations for the structural design of pavements with a bituminous surfacing. The pavement designs are primarily based on the results of full-scale experiments and studies of the performance of existing roads in the tropics. The designs can be used for roads with traffic load of up to 30 million so-called cumulative equivalent standard axles in one direction during the design period. For more heavily trafficked roads, it is recommended in the Road Note that the design method used in the United Kingdom (ref. 3) be applied. This method, however, is likely to require calibration to the conditions encountered in the tropics.

16.2.2 Terminal condition

The condition of the pavement by the end of the design period is not defined in the Road Note. It is indicated that for the recommended pavement structure, 'the level of deterioration that is reached by the end of the design period has been

restricted to levels that experience has shown give rise to acceptable economic designs under a wide range of conditions'. It is assumed that 'routine and periodic maintenance activities are carried out to a reasonable, though not excessive, level'. Finally, it is stated that 'design life does not mean that at the end of the period the pavement will be completely worn out and in need of reconstruction; it means that towards the end of the period the pavement will need to be strengthened so that it can continue to carry traffic satisfactorily for a further period'. What is meant by 'satisfactorily' is not explained. It is evident that the method does not allow prediction of the gradual deterioration.

16.2.3 Traffic

The loads imposed by passenger cars do not contribute significantly to the structural damage caused to the road pavements by traffic. Therefore, for the purpose of pavement design, private cars are ignored and only the total number and the axle loading of the heavy vehicles that will use the road during the design life are considered. In this context heavy vehicles are defined as those having an unladen weight of 3000 kg or more.

Number of vehicles

In order to estimate the total number of commercial vehicles that will traverse the pavement in the course of its design life, it is necessary to:

- know the number of heavy vehicles that will use the road the first year the road is open;
- forecast the annual growth rate of this traffic;
- select the design life.

Estimation of the initial traffic and forecast of the traffic growth are outlined in Chapter 3.

Design life

The design life for road pavements is commonly selected to be between 10 and 20 years. A design life of 15 years is recommended because it reduces the problem of forecasting uncertain traffic trends. In the previous edition of Road Note 31, 'stage construction' was recommended for roads with a high anticipated traffic growth. Stage construction consists of planned strengthening of the pavement after maybe 10 years. In the new edition of the Road Note, stage construction is not advocated as a general policy because experience has shown that budget constraints often prevent planned strengthening.

Axle loads

The axle load distribution and damaging effect of heavy vehicles may vary considerably from one country to another. There may also be marked differences between the axle loads of vehicles using different classes of roads in a country. When a major road is being designed, it is recommended that an axle load survey of heavy vehicles is undertaken in the area where the road will be constructed and of the class of road in question unless recent data are available.

Road Note 31 relates the damaging effect of axle loads to a 'standard axle' of 80 kN using a conversion model which has been derived from the AASHO road test (ref. 1):

$$N_{esa} = \left(\frac{P}{80}\right)^{4.5} \cdot N_p$$

where:

N_{esa} = number of equivalent standard axles;

N_P = number of axles with load P kN.

The total number of heavy vehicles that will use the road during its design life is converted to a cumulative number of equivalent standard axles (CN_{esa}).

The pavement design should be based on the cumulative number of standard axles in the busiest lane of the road. For two-lane roads with two-way traffic and no significant difference between the two traffic streams, the design number of standard axles is assumed to be 50% of the total number of standard axles in the two directions.

16.2.4 Subgrade strength

The strength of the subgrade is assessed in terms of the California bearing ratio (CBR). CBR

Road Note 31 deals more closely with the influence of water on the subgrade strength than other pavement design methods. For estimation of the design moisture content, the subgrade moisture conditions under impermeable asphalt surfacings are classified into three categories.

In category 1 the water table is sufficiently close to the ground surface to control the subgrade moisture content. The type of subgrade soil governs the depth below the road surface at which a water table has a dominant influence. For non-plastic soils, sandy clays and heavy clays the water table will dominate the subgrade moisture content when the distance from the road surface is less than 1 m, 3 m, and 7 m respectively. Category 1

The best and easiest method of evaluating the design moisture content is to take measurements below similar, existing pavements during the wet season. If there is no existing road in the vicinity, the design moisture content can be estimated from measurements of the depth of the water table and determination of the relation between suction and moisture for the subgrade soil. A test apparatus for determining this relationship is described in the Road Note.

In category 2 the water table is deep but the rainfall is sufficient to produce significant seasonal changes in the moisture conditions under the road. This situation occurs when rainfall exceeds evapotranspiration for at least two months of the year. The rainfall in such areas is usually greater than 250 mm per year and is often seasonal. In this case the subgrade moisture condition will depend on the balance between the water entering the subgrade through the shoulders and the moisture leaving the ground by evapotranspiration when the weather is dry. The design moisture content should be taken as the optimum moisture content given by the standard Proctor compaction test. Category 2

Although it is assumed that the surfacing is impermeable, saturated subgrade conditions may be anticipated in some cases in both categories 1 and 2. In such cases the subgrade strength should be assessed based on saturated CBR samples. Exception

In category 3 the permanent water table is deep, and the climate is arid throughout the year. Such areas have an annual rainfall of less than 250 mm. The moisture content of the subgrade is unlikely to exceed the optimum moisture content given by the standard compaction test and this moisture content should be used for design purposes. Category 3

CBR tests

The compaction properties of the subgrade soil are determined by carrying out standard compaction tests in the laboratory. Next, samples are compacted in CBR moulds to 100% of the maximum dry density achieved in the compaction tests. The CBR samples are left sealed for 24 hours before being tested in order to remove any pore water pressure induced during compaction. Saturated CBR samples are obtained by immersing the compacted samples in water for four days before being tested.

Design CBR

The Road Note gives no indication of the number of samples that should be tested. Certainly, the number depends on the variability of the subgrade and groundwater level. The samples will usually have different strengths. According to the Road Note the design CBR is taken as the value that is exceeded by 90% of the test results. If the test results change significantly over sections of the road, a separate design strength should be calculated for each section.

If equipment for performing CBR tests is not available, a less precise estimate of the design subgrade strength can be obtained from Table 16.1.

Table 16.1 Estimated minimum design CBR values (TRL Overseas Road Note 31, ref. 2). The table is not applicable for silt, micaceous, organic or weathered clays.

Depth of water table from formation level[1]	*Minimum* CBR *(%)*				
	Non-plastic sand	*Sandy clay PI = 10*	*Sandy clay PI = 20*	*Silty clay PI = 30*	*Heavy clay PI > 40*
0.5 m	8–14	8–14	3–4	3–4	2
1.0 m	15–29	8–14	5–7	3–4	2
2.0 m	15–29	15–29	8–14	5–7	3–4
3.0 m	≥30	15–29	8–14	5–7	3–4

[1] The highest seasonal level attained by the water table should be taken.

Dynamic cone

In areas where an existing road is placed on the same type of subgrade as the new planned road and the water table is located at the same depth, direct measurement of the subgrade strength may be made by use of a dynamic cone penetrometer.

16.2.5 Pavement materials

The different types of materials that may be used for wearing course, base, subbase and capping layers are defined in Chapters 14 and 15.

16.2.6 Design

Because of the large uncertainties of forecasting traffic loads and the large variability in material properties, the designs are presented as a catalogue of a number of different pavement structures. Each structure in the catalogue is applicable to a small range of traffic loads and subgrade strength.

Capping layer

A capping layer is used on subgrades with a design CBR of 2–4 and in some

cases also on subgrades with a design CBR of 5–7. For subgrades with CBR less than 2, special treatment is required, which is not covered by the Road Note.

Figures 16.3 and 16.4 show two design charts from the catalogue. A key to the charts is given in Figure 16.2. Input in the design charts are the cumulative number of equivalent standard axles (CN_{esa}) and the subgrade design CBR.

Design charts

16.3 THE SATCC DESIGN GUIDE

16.3.1 Background

The SATCC design guide (ref. 4) was developed for the Southern Africa Transport and Communication Commission for use in Angola, Zambia, Botswana, Zimbabwe, Mozambique, Malawi, Swaziland, Lesotho, and Tanzania. For medium to heavy traffic the design method is based on an old AASHTO design

Traffic class	10^6 CN_{esa}	Subgrade class	CBR %
T1	< 0.3	S1	2
T2	0.3 - 0.7	S2	3 - 4
T3	0.7 - 1.5	S3	5 - 7
T4	1.5 - 3.0	S4	8 - 14
T5	3.0 - 6.0	S5	15 - 29
T6	6.0 - 10	S6	30+
T7	10 - 17		
T8	17 - 30		

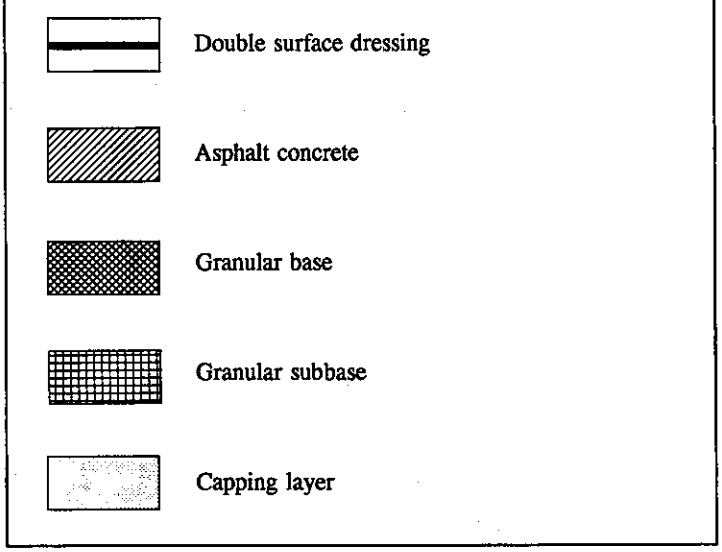
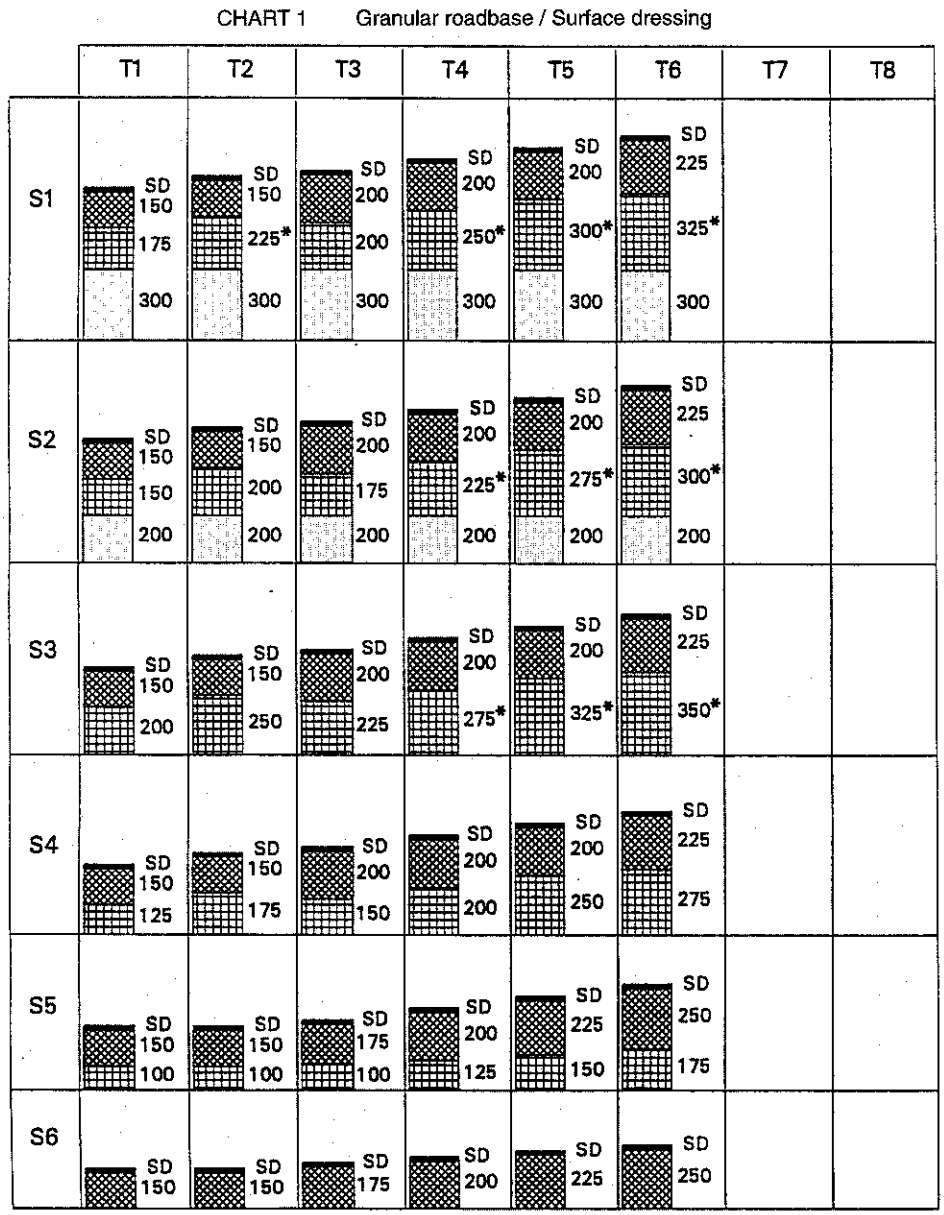


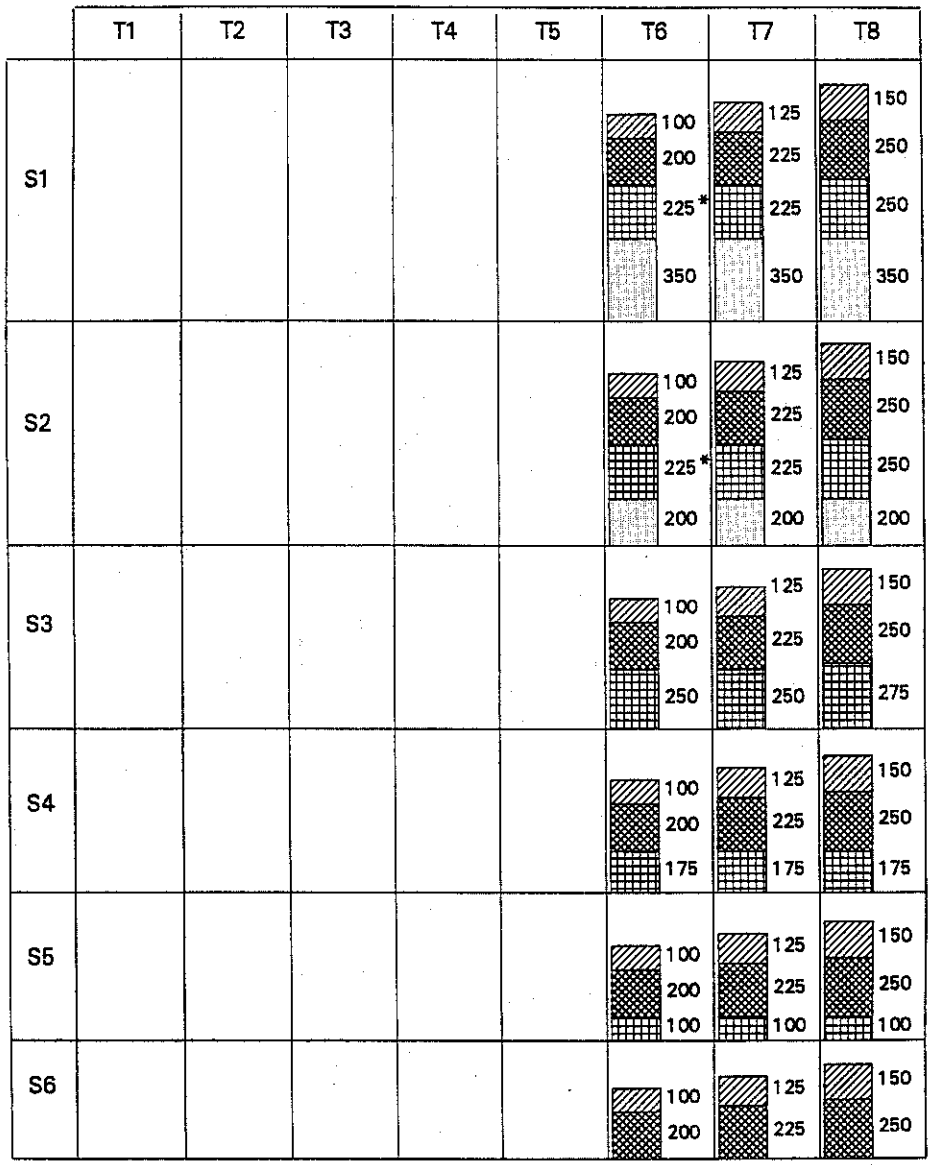
Figure 16.2 Key to the design charts in TRL Overseas Road Note 31 (ref. 2).



Note: 1 *Up to 100 mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200 mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25 mm : 32 mm.
2 A cement or lime-stabilised sub-base may also be used.

Figure 16.3 Design chart from TRL Overseas Road Note 31 (ref. 2).

CHART 5 Granular roadbase / Structural surface



Note: 1 * Up to 100 mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200 mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25 mm : 32 mm.
2 A cement or lime-stabilised sub-base may also be used.

Figure 16.4 Design chart from TRL Overseas Road Note 31 (ref. 2).

equation (ref. 5), whereas for light traffic an old edition of Road Note 31 was taken into account. Criteria for adjustment of the so-called layer coefficients to local conditions are based on findings from different research projects including the preparatory studies for the World Bank Highway Design and Maintenance standards model.

16.3.2 Terminal condition

PSI

The pavement condition is expressed by the present serviceability index (PSI). As part of the AASHO road test (ref. 1) a broad panel of road users was asked to drive across a great number of different roads and indicate the functional conditions on a marking scale between 0 and 5. A rating of 5 was given to roads in a perfect condition, whereas 0 was given to roads in so poor a condition that the driver would not like to drive at normal speed. The average rating obtained for each road was called the present serviceability rating (PSR). The PSR was then correlated with objective measurements of road roughness, rutting, cracking and patching. The roughness turned out to be the dominating factor. The objective measure is called the present serviceability index (PSI) in order to distinguish it from the subjective PSR.

Terminal PSI

In the AASHTO design guide a PSI of 2.0 is recommended as the terminal condition for rural roads in USA. This value is based on a 20-year design period without maintenance. In the SATCC guide a PSI of 1.5 has been found adequate as the terminal condition, assuming adequate periodic maintenance of the pavement.

The SATCC guide does not explain how the gradual deterioration of a pavement can be predicted although it is possible to use the AASHTO design guide to predict the functional deterioration over time.

16.3.3 Traffic

Number of vehicles

The number of commercial vehicles of different types that will use the road during its design life is estimated as previously described.

Design life

For the types of pavement proposed in the guide the initial design period for flexible pavements should not exceed 15 years and for semi-flexible pavements 20 years. Stage construction is recommended if this type of design offers economic advantages.

Axle loads

The axle loading of different vehicle types is converted to a number of equivalent 80 kN standard axles (N_{esa}) by use of the same equation as applied in Road Note 31. However, an exponent of 4.0 instead of 4.5 is recommended in the SATCC guide. Table 16.2 shows some average values of N_{esa} for different types of vehicle.

Table 16.2 Average number of equivalent 80 kN standard axles (ref. 4).

Vehicle type	*Nesa*
Rigid truck with drawbar trailer	6.0
Articulated unit with semi-trailer	6.0
Two-axle and tandem axle rigid truck	2.0
Bus	1.0

These figures may be used for the feasibility study stage and for secondary projects.

The total number of commercial vehicles that will use the most severely loaded lane of the road during its design life is converted to a cumulative number of equivalent standard axles (CN_{esa}).

16.3.4 Subgrade strength

CBR

The bearing capacity of the subgrade is assessed from the CBR of representative soil samples. In all climates other than the very arid, the CBR tests should normally be performed on samples compacted to the minimum density required for the subgrade and soaked in water for four days. The SATCC guide does not indicate how the design CBR should be derived from different test values.

Capping layer

Subgrades with a CBR of less than 2% are considered unsuitable and should normally be replaced. If the CBR is 2–7%, special measures should be taken to strengthen the subgrade before the subbase is placed. This strengthening corresponds to the capping layer applied in Road Note 31. The strengthening is accomplished by placing a 200–250 mm thick compacted layer of gravel or selected fill or a 150 mm thick layer of lime or cement-stabilized material on the subgrade. It is assumed that this improves the subgrade design CBR to 8%.

16.3.5 Pavement materials

Materials that, according to the SATCC guide, are applicable for subbase, base and wearing course are listed in Table 16.3. The most notable difference between the selection of materials in the table and in Road Note 31 (cf. Chapter 15) is that rolled asphalt and sand asphalt only occur in the Road Note. The SATCC guide defines the following rules for combining different materials into a pavement structure.

Subbase

A subbase of natural gravel can be combined with any type of base. A subbase of cement or lime-stabilized material requires a base of graded crushed stone or bituminous macadam.

Base

A base consisting of high-strength cement stabilized gravel or bituminous macadam is only recommended for pavements with heavy traffic.

Wearing course

Double surface dressing is recommended as wearing course for roads with CN_{esa} up to 2.5 million. Asphalt concrete is recommended as wearing course for roads carrying more heavy traffic. Single surface dressing should only be used as wearing course when the base is made of asphalt.

Layer coefficients

The 'bearing capacity' of different pavement materials is indicated by 'layer coefficients' (Table 16.3). The concept of layer coefficient may be explained by an example. The layer coefficient of bituminous macadam and natural gravel is 0.20 and 0.12 respectively. This means that 10 mm of bituminous macadam are equal to $10 \times (0.20/0.12)$ mm = 17 mm of gravel. The layer coefficients are used in the design model as explained in the following section.

16.3.6 Design

Structural number

The required bearing capacity of the pavement is expressed as a 'structural number' (SN). For CN_{esa} higher than 0.5×10^6, the required SN is calculated from the empirical equation:

Table 16.3 Materials applicable for subbase, base and wearing course (ref. 4).

Layer/material	*Layer coefficient*	*Thickness range (mm)*
Wearing course:		
Single surface dressing[1]	$\alpha_1 = 0.20$	–
Double surface dressing[1]	$\alpha_1 = 0.20$	–
Asphalt concrete[2]	$\alpha_1 = 0.35$	30–90
Base:		
Asphalt macadam	$\alpha_2 = 0.20$	70–150
Gravel (CBR > 80%):		
natural	$\alpha_2 = 0.12$	125–200
crushed	$\alpha_2 = 0.13$	125–200
Graded crushed stone (CBR > 100%)		
on natural gravel subbase	$\alpha_2 = 0.14$	125–200
on stabilized subbase	$\alpha_2 = 0.18$	125–200
Cement-stabilized gravel:		
with UCS = 2.0–3.5 MPa	$\alpha_2 = 0.14$	125–200
with UCS = 3.5–5.0 MPa	$\alpha_2 = 0.18$	125–175
Subbase:		
Natural gravel (CBR > 30%)	$\alpha_3 = 0.11$	100–250
Cement or lime-stabilized materials:		
with UCS = 0.7–2.0 MPa	$\alpha_3 = 0.12$	100–200
with UCS > 2.0 MPa	$\alpha_3 = 0.16$	100–200

UCS = unconfined compressive strength after seven days.
[1] Surface dressing is assumed to contribute to the strength of the pavement only when applied on a bituminous layer to improve impermeability of the wearing course.
[2] Adjustment for extreme temperature conditions is described in the design guide.

$$\log CN_{esa} = 9.36 \log\left(\frac{SN}{25.4} + 1\right) - 0.83 + \log\frac{1}{R} + 1.395 \log CBR$$

where:

SN = structural number in mm;

CBR = Design CBR for the subgrade (minimum 8%);

R = regional adjustment factor.

Regional adjustment

The regional adjustment factor R makes the design equation applicable to design of pavements in areas with different climatic conditions. R is assumed to be 1.0 for areas with rainfall throughout most of the year, creating permanently saturated conditions of the subgrade and the unbound pavement layers (12 wet months). A factor of 0.1 is assumed for very arid climates (0 wet months) where the pavement structure and the subbase never reach a saturated condition.

For CN_{esa} less than 0.5×10^6 no equation is given for SN, but different values of SN are listed for different ranges of subgrade CBR (ref. 4).

Thickness design

After the materials for the different pavement layers have been selected the thicknesses are determined by trial and error so that the following equation is satisfied:

$$a_1 \cdot h_1 + a_2 \cdot h_2 + a_3 \cdot h_3 = SN$$

where a_1, a_2, and a_3 are the layer coefficients of the wearing course, base and subbase material, respectively; and h_1, h_2, and h_3 are the thickness in mm of the wearing course, base and subbase respectively.

In the case of a fourth pavement layer the equation is extended by adding a_4 and h_4.

16.4 THE ANALYTICAL–EMPIRICAL METHOD

16.4.1 Background

Industrialized countries

The analytical–empirical method (ref. 6) was introduced in Denmark in the early 1960s. Shell, the world-wide producer of bitumen, also adopted the method quite early, and during the last few years many countries have started using the method.

Advantages

The analytical–empirical method has some obvious advantages, when compared to totally empirical methods:

- the properties of existing pavements can be measured *in situ* by a non-destructive method, which is extremely useful for maintenance planning and design of overlays (cf. a later section in this chapter);
- the properties of the materials may be measured for all climatic conditions and used for the calculation of stresses and strains;
- stresses and strains may be calculated for all combinations of axle loads, tyre configuration and tyre pressure;
- the deterioration of a pavement can be related to the deterioration of each pavement layer;
- prediction of gradual deterioration is possible.

Developing countries

The analytical–empirical method has been introduced in some Asian countries. Malaysia and the Philippines are using the method as a routine for investigation (management) of existing pavements on highways with heavy traffic. Indonesia is presently using the method for research and Thailand and China are in the early stages. In some African countries consultants have used the method for big, isolated projects. However, the introduction of the analytical–empirical method as a standard is taking longer than expected in most developing countries. The reason is that in addition to calibration according to local conditions, the method requires rather sophisticated test equipment.

Permissible stresses

As indicated by the name the design has two steps. The first step is the calculation of the permissible stress in each pavement layer, i.e. the stress that will cause the structural and the functional conditions to deteriorate to the terminal conditions. This is achieved by using an empirical relationship between the stress, the number of repeated stresses, and the rate of deterioration. The empirical relation used in Asian countries is mostly based on Australian experience.

Actual stresses

The second step of the design is the calculation of the actual stress occurring in each pavement layer. This stress is calculated using an analytical method, e.g. the theory of elasticity. The calculations are repeated for all important combinations of loading and environmental conditions.

Linear elasticity

When the theory of elasticity is applied, it is assumed that the subgrade and pavement materials are homogeneous, isotropic and linearly elastic. The elastic

properties are expressed as the E moduli of the materials. According to Hooke's law for the uniaxial case:

$$E = \frac{\sigma}{\varepsilon}$$

where:

σ = stress;
ε = strain.

Falling weight deflectometer

The E modulus may be measured in the field by use of a falling weight deflectometer (FWD) as shown in Figure 16.5. The FWD simulates the dynamic wheel load from a moving truck. The deflection basin under the load is measured by placing a number of geophones at different distances from the centre of the load. If the thickness of each pavement layer is known, it is possible to calculate the elastic modulus of each pavement layer. It is beyond the scope of this book to explain how, but the calculations are quite simple if an approximate method is applied.

Requirements

Two requirements should be met when the elastic moduli are computed from the shape of the deflection basin. First, it is important that the test load resembles that of a heavy wheel load, both in size and in duration. Pavement materials do not comply very well with the assumption that they are homogeneous, isotropic and linear elastic. As a consequence the apparent elastic moduli vary with the stress conditions. The stresses created by the FWD match those under heavy traffic loads. This is not the case when using other types of dynamic test equipment, e.g. light-weight vibrators or wave propagation methods.

The second requirement is that the deflection should be measured very accurately, especially at points at some distance from the centre of the load. Extreme accuracy is obtained by use of geophones because geophones do not need any reference. The 'reference' is the centre of gravity of the earth. For many

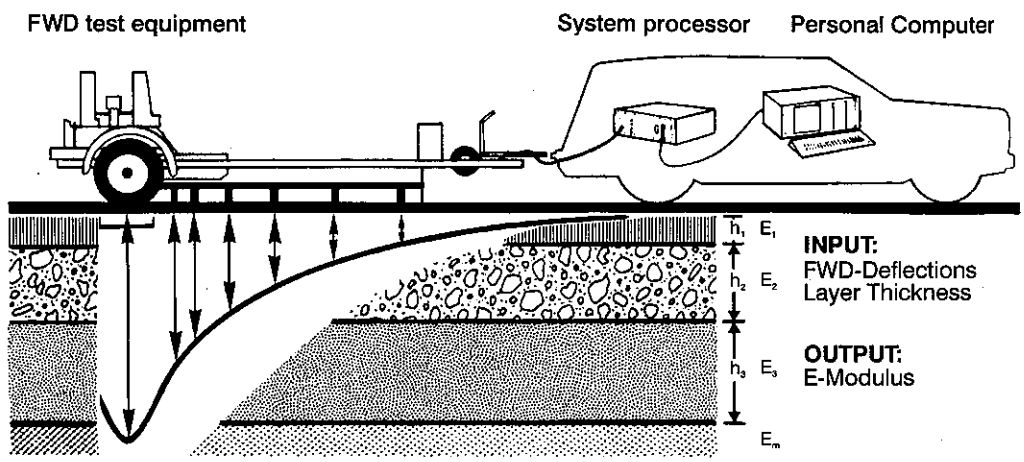


Figure 16.5 Falling weight deflectometer.

reasons the accuracy of the Benkelman Beam is not suitable for use in connection with the analytical–empirical design method.

In the following section the Danish analytical–empirical design method is illustrated. In this method the analytical part is based on elasticity theory.

16.4.2 Terminal conditions

Two terminal conditions are defined. One for premixed asphalt and cement bound materials, and one for unbound materials including subgrade soils.

Bound materials

Premixed asphalt and cement bound materials are supposed to deteriorate due to cracking, i.e. structural deterioration. The terminal condition is defined as a certain amount of cracking.

Unbound materials

Unbound materials are supposed to deteriorate due to plastic deformations, i.e. functional deterioration. The terminal condition is defined indirectly as a deterioration corresponding to a present serviceability index of 2.0–2.5.

It is possible to predict the gradual deterioration of a pavement. To do this, the empirical relationships must be changed to predict the rate of distress, at any point in time, rather than the number of loads to cause a certain amount of distress. In other words, for unbound materials the relationships should give the increase in permanent deformation per load at a given vertical, compressive stress in each layer.

16.4.3 Traffic

CN_{esa}

The number of vehicles of different types that will use the road during the design life is estimated as previously described. The design method imposes no restriction on the design life. The axle loadings of different vehicles are converted to a number of equivalent standard axles using the AASHO equation and an exponent of 4.0. Any axle load could be taken as the standard axle. In Denmark 100 kN is used. Finally, the cumulative number of equivalent standard axles (CN_{esa}) in the most severely loaded lane is calculated.

16.4.4 Subgrade strength

E modulus

When designing strengthening layers on existing roads, the *E* modulus of the subgrade is measured with a FWD. For new roads it is normally not practical to use FWD surveys. Instead, the *E* modulus may be estimated from CBR tests using the following equation:

$$E = 10 \cdot \mathrm{CBR}.$$

However, the accuracy of this equation, relating a deformation parameter to a strength parameter, is disputed.

16.4.5 Pavement materials

E moduli

For pavement materials it is common practice to use previously established standard values for the *E* moduli. The standard values have been derived from a great number of FWD measurements on existing pavements. As an example,

Table 16.4 Standard E moduli for some Danish pavement materials.

Material type	*E modulus (MPa)*
Asphalt concrete	2000–3000
Bituminous macadam	2000–5000
Macadam	1000
Graded, natural gravel	300
Natural gravel	100–150

Table 16.4 shows the standard E moduli assigned to some Danish pavement materials.

16.4.6 Design

Only thickness design of unbound materials will be discussed in this book. For design of premixed asphalt and cement-bound materials reference is made to ref. (6).

Compressive stress

Deformation of unbound materials occurs over time by accumulation of numerous, diminutive, plastic deformations. These deformations are caused by traffic-induced vertical, compressive stresses in the materials. According to the elastic layer theory, the maximum vertical stress in a layer occurs on the surface of the layer.

Permissible stress

The permissible (allowable), vertical stress on an unbound layer is calculated from an empirical equation, based on the AASHO road test and Danish experience:

$$\sigma_{per} = 8.34 \cdot CN_{esa}^{-0.307} \cdot \left(\frac{E}{160}\right)^{\alpha}$$

where:

σ_{per} = permissible vertical stress;
CN_{esa} = cumulative number of equivalent 100 kN axle loads;
E = E modulus of the material;
α = 1.16 for $E < 160$ MPa and 1.00 for $E > 160$ MPa.

The permissible stress increases with the E modulus and decreases with the cumulative number of equivalent standard axles. The equation would need calibration in order to apply to pavement types and weather conditions different from what is found in North America and Denmark.

Actual stress

The next step is to calculate the stresses that actually occur under an equivalent wheel load. The equivalent wheel load is half the equivalent axle load. Commonly, the wheel load is increased by 20% as a safety measure. The contact pressure is normally put equal to the maximum tyre inflation pressure. The wheel load is assumed to be uniformly distributed over a circular area.

If the equivalent wheel load plus a 20% addition is 60 kN and the tyre pressure is 0.7 MPa, then the radius in the contact area can be calculated from:

$$\sigma_0 \cdot \pi \cdot a^2 = P$$

where:

σ_0 = contact pressure;
a = radius in the contact area;
P = wheel load.

One layer

If the load is placed on the surface of a homogeneous, linear elastic and isotropic material, the vertical compressive stress in any depth under the centre of the load can be calculated from an equation developed by Boussinesq in 1885:

$$\sigma_z = \sigma_0 \left(1 - \frac{1}{(1 + (a/z)^2)^{3/2}}\right)$$

where:

z = depth below the surface;
σ_z = vertical stress at depth z.

Multiple layers

For multiple layers, computer models have been developed for the vertical stresses at any depth below the surface. However, the computations are complicated and require relatively large amounts of computer time. This may not be justified, when it is considered that the assumptions made of continuous, homogeneous, isotropic and linear elastic materials are not true for pavement materials. Instead, an approximative method known as the method of equivalent thicknesses (MET) may be used. The method can only be applied if the E modulus of every layer in the system is higher than the E modulus of the layer below. The MET should not be confused with the equivalence factors used in the SATCC design method.

Constant stiffness

The principle of MET is as follows. When calculating stresses at an interface between two layers, all layers above the interface are transformed into layers with the same E modulus as the layer below the interface. This is accomplished by increasing the thickness of each layer above the interface in such a way that they keep their stiffness. It is assumed that the stiffness of the layer remains constant if the following quantity is kept constant:

$$h^3 \cdot E$$

where:

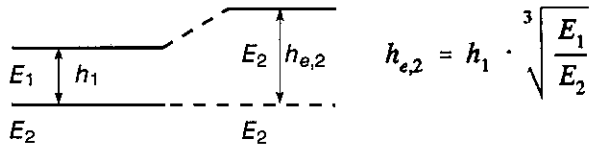
h = thickness of the layer;
E = E modulus of the material.

Equivalent thickness

Now consider the two-layer system in Figure 16.6. If E_1 is changed to E_2 then h_1 should be changed to the equivalent thickness $h_{e,2}$ derived from the equation given in the figure.

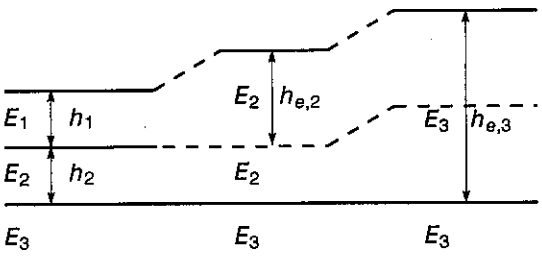
Step-wise transformation

The equations for the step-wise transformation to equivalent thickness for a three- and four-layer system are given in Figure 16.7. f is a correction factor introduced in order to obtain better agreement with the exact elastic theory (exact in a mathematical sense, not in a physical sense). A frequently used value for f is 0.8. No correction factor is used for the first interface.



$$h_{e,2} = h_1 \cdot \sqrt[3]{\frac{E_1}{E_2}}$$

Figure 16.6 Two-layer system.



$$h_{e,3} = f \cdot (h_1 \cdot \sqrt[3]{\frac{E_1}{E_2}} + h_2) \cdot \sqrt[3]{\frac{E_2}{E_3}}$$

$$h_{e,4} = f \cdot [(h_1 \cdot \sqrt[3]{\frac{E_1}{E_2}} + h_2) \cdot \sqrt[3]{\frac{E_2}{E_3}} + h_3] \cdot \sqrt[3]{\frac{E_3}{E_4}}$$

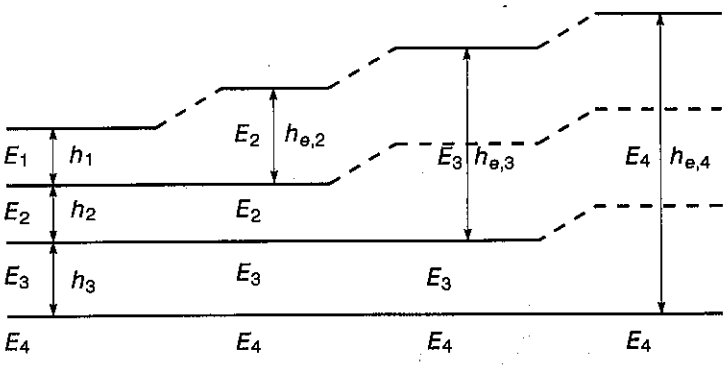


Figure 16.7 Three- and four-layer systems.

Compressive stress

Finally, the actual vertical compressive stress on top of each unbound layer is calculated from the equation developed by Boussinesq for a homogeneous, linear elastic and isotropic material. The stresses are compared with the permissible stresses. If the stresses are higher or much lower than permissible, then the thicknesses of the pavement layers are adjusted and the stresses in the new pavement structure calculated. The procedure is repeated until the calculated stresses are equal to or somewhat lower than permissible.

16.5 DESIGN OF GRAVEL PAVEMENTS

Overseas Road Note 31, the SATCC design guide, and most other pavement design methods only deal with bituminous surfaced roads or concrete roads. In principle, the analytical–empirical methods can be used for all types of pavements.

In developing countries a substantial part of the road network is surfaced with

earth or gravel. Most rural access roads are likely to remain unsealed for many years, and many new gravel roads will be needed in the future.

Thickness estimation

Normally, the thickness of a gravel surfacing is not designed. The thickness is simply estimated based on experience. Gravel is typically placed in a compacted thickness of 150–200 mm (ref. 10). In areas where the thickness appears to be inadequate resulting in rapid formation of deep wheel tracks, more gravel is added.

Drainage

New gravel roads are often constructed along the alignment of old tracks or earth roads. Before upgrading an old track or earth road to a gravel road, careful attention should be given to the drainage. Alignments following valley bottoms subject to flooding during the wet season should be relocated to higher levels. Road-side ditches, turn-outs, and culverts should be provided where necessary. Many examples are known from the tropics where new gravel roads have been washed away during the first rainy season due to missing or inadequate drainage.

Sealing of gravel roads

Gravel roads are dusty in the dry season. They need frequent maintenance to avoid development of corrugations and potholes. When traffic reaches a certain level, the question of supplying the surface with a bituminous seal may arise. However, also a surface dressing needs maintenance. An investment model (cf. Chapter 4) could be used to decide when the sum of road maintenance and user costs is less for a surface dressing than for a gravel surface. However, if it is likely that a future asphalt surfacing will not receive adequate maintenance, it may be advisable to keep on to the gravel surface. Also it should be remembered that a surface dressing should not be applied directly to an old, worn gravel surface. Normally, it is necessary first to place a new layer of fresh gravel.

16.6 STRENGTHENING

When a pavement is nearing the end of its design life, it is usual to strengthen it by adding an overlay. This is true whether the existing pavement was initially prepared for stage construction or not.

16.6.1 The SATCC guide

Strengthening is not discussed, neither in Road Note 31 nor in the SATCC guide. However, it is straightforward to use the guide for design of overlays on existing pavements.

Examinations

For overlay design the testing of the subgrade is supplemented by a thorough examination of the old pavement. The thickness of each layer of the old pavement is recorded and the materials tested in the field or in the laboratory. Based on the tests each layer is assigned a layer coefficient. A cracked bituminous macadam may be given the same layer coefficient as a gravel base. A disintegrated gravel base may be given the same coefficient as a gravel subbase and a decomposed subbase material may be disregarded and treated as part of the existing subgrade.

Thickness design

A new pavement is then designed with the old pavement incorporated as one or two lower layers with given thicknesses. Strengthening of the old pavement is achieved by placing one or more overlays on top of the old pavement, e.g. an asphalt concrete surfacing or a base-course with a double surface dressing.

16.6.2 The deflection method

A widely used method for designing overlays is based on the use of Benkelman beam deflection measurements in assessing the strength of existing pavements (refs 8 and 9).

Benkelman beam

A Benkelman beam deflection measurement is a simple load test, where the dual, rear wheels of a heavy truck are used as the load (ref. 10). The Benkelman beam is a 3–4 metre long, slender pivoted arm which can be placed between the dual wheels. The pivot is two-thirds of the distance from one end (the toe of the beam) and is carried by a frame resting on the road as shown in Figure 16.8. The toe of the beam is placed resting on the pavement in the gap between the walls of the dual wheels. The truck is moved away and the vertical displacement of the other end of the beam is recorded. The recorded movement multiplied by two is equal to the rebound deflection of the pavement under the load from the dual wheels.

The Benkelman beam measurements should be made at the time of the year when the pavement is at its weakest, i.e. its wettest condition. The measurements should be made in the wheel-tracks and at intervals along the road of not more than 100 metres. The road is then divided into sections in which the deflection measurements are reasonably constant. The minimum length of a section should be compatible with the intended method of resurfacing. The 'design deflection' for each road section is calculated as the mean plus 1.5 times the standard deviation.

Thickness design

The necessary thickness of a strengthening overlay is read from design charts based on experience. Figure 16.9 shows an overlay design chart for pavements with granular road bases and overlays of rolled-asphalt. This chart has been found satisfactory for tropical conditions. The deflection should be measured with a dual wheel load of 31 kN and a tyre pressure of 0.6 MPa. The cumulative number of equivalent standard axles refers to a 80 kN standard axle.

Drainage

At this stage of the design process the condition of the road drainage should be reviewed and, where deficiencies have weakened the existing pavement, the road drainage should be improved. Improvements to the drainage system should be made before adding the overlay.

Shortcomings

The deflection beam is simple to use and maintain and thus has strong attractions for use in many countries where the operation of automated equipment can

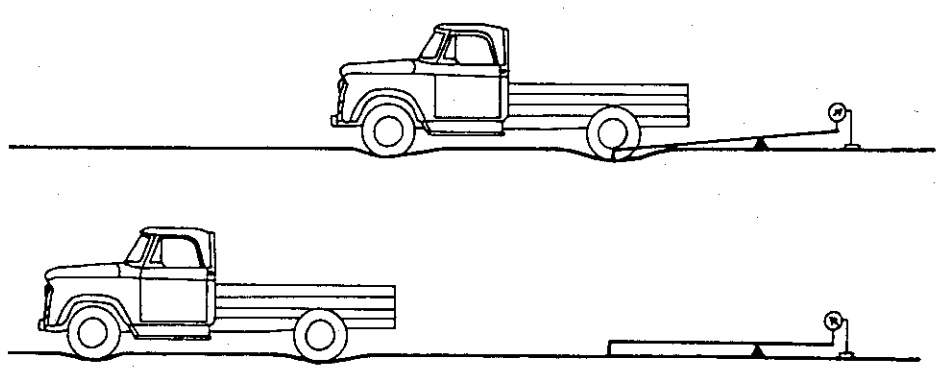


Figure 16.8 Benkelman beam measurement.

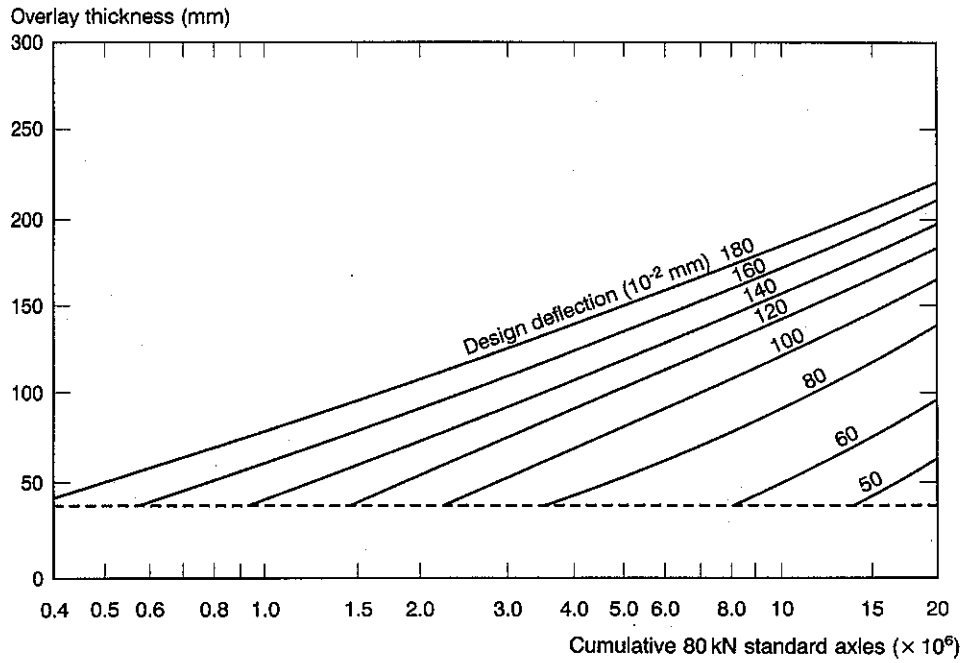


Figure 16.9 Overlay design chart for pavements with granular base (ref. 9).

involve high operating and maintenance costs. However, design of overlays based only on the surface deflection of the existing pavement surface under the centre of a static load (and the number of expected standard axles) is subject to great uncertainty. The reason is that the stresses and strains induced in the pavement structure and the subgrade in different depths below the surface are not only a function of the depth and the surface deflection, but also of the thickness and stiffness of the pavement layers. Two pavements having the same surface deflection may have very different bearing capacities.

16.6.3 The analytical–empirical method

As previously explained the analytical–empirical method is well suited for design of overlays. Using a FWD survey, it is possible to evaluate the structural condition of an existing road. The E moduli of the subgrade and each existing pavement layer are calculated based on the deflections recorded at different distances from the centre of the falling weight. A material for the overlay is selected, and the required thickness is calculated in a way similar to that described for new pavements. The difference is that the E moduli and the thicknesses of the existing pavement layers are fixed. Only the thickness of the overlay is designed.

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PART 7

Construction



Labour-intensive road construction in Kenya. (Photo by Jorgen Schytte)

17 Construction contracts

Tim Waage, Kampsax International

17.1 PROJECT EXECUTION METHODS

The design and construction of roads can be undertaken in different ways. Normally, one of the following methods is adopted:

- in-house project execution;
- turnkey projects;
- BOT projects;
- project management;
- construction contracts.

In-house

In-house project execution presupposes that all aspects of planning and execution of a road project are handled exclusively by one party; normally a central or regional highway authority, municipality and the like. As each and every step is handled by the Owner's organization, there is no possibility of conflict with outside parties, and the personnel involved are familiar with the requirements, procedures and policies related to the project. It is often possible to save time by the in-house method because construction can commence before the design is completed. A disadvantage is that adequate internal cost control is difficult to implement, and this tends to result in higher overall project costs than other project execution methods. There is also a clear tendency towards less strict quality control (why check oneself), and there is less incentive to develop more efficient design and construction technologies. Another problem is that it is difficult for the Owner, being a government or public body, to comply with labour laws and regulations in the face of annual fluctuations in the number and composition of workers required. Finally, for developing countries it is a major drawback that international aid and lending agencies tend not to favour the in-house method. In fact, the last 20 years have seen a clear tendency in most industrialized countries towards other project execution methods.

Turnkey

Under the turnkey method the Owner enters into a contract with a single company or entity for the purpose of both design and construction of the complete

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project. Such a company will often be required to provide or acquire the necessary project financing. Advantages of turnkey projects are that the Owner has to deal with one responsible party only, and total project costs will be known before a final decision is made to go ahead with the scheme. Another plus is that construction may be started before design has been finalized, which may advance project completion. Disadvantages are that it can be complicated to define the terms or content of a turnkey project. This means that it may be difficult to select a company on a competitive basis, as the various companies may offer widely differing terms and conditions. Errors are not easily detected and the Owner's influence on project matters is less pronounced than for other project execution methods. The turnkey solution is best suited for projects which are relatively straightforward and well defined, for example standardized housing and industrial complexes. It is widely used for projects where specialized expertise is held by a few organizations.

BOT

'Build, operate and transfer (BOT) projects' are a later and more refined version of the turnkey method. In BOT projects private-sector sponsors, usually international contractors, suppliers and other interested parties together with local partners, including government agencies, make equity investments in a private project company established specifically to develop and implement the project, operate it long enough to pay back the debt and equity investment, and then transfer the project to the host government. An example of a contractual structure for a BOT project is shown in Figure 17.1. The BOT projects tend to be complex, time and effort consuming, costly to develop and risky. Advantages are that private sector financial, management and entrepreneurial inputs are added to those of the government to realize more projects. Lessons from around the world have shown that to minimize risks BOT projects should be done in a simplified manner, by a highly knowledgeable, small group, with maximum project preparation.

Project management

The project management method is best suited for projects which are complex and require a tight schedule. Within the highway sector it would normally be considered only for very large bridge projects. Under this type of project execution the Owner appoints a firm of consulting engineers to administer the design and construction on his behalf. The duties and responsibilities of the consulting engineers will range from project planning and design through procurement services to project management and supervision. Construction is generally handled by a number of specialized contractors, supervised by the appointed consulting engineering company. The advantage of the project management method is that the Owner will have the benefit of expert professional advice at all stages of the project. By implementing a counterpart system of foreign experts and local engineers in developing countries a high degree of transfer of knowledge can be achieved. The method is particularly well suited to ensuring that tight construction schedules can be met, and all construction and supply contracts can be awarded on the basis of competitive bids for well-defined services.

As the individual contracts can be of limited size the method makes it easier to employ small local contractors for the purpose of developing their skills and competence. A main disadvantage of the project management method is that it tends to complicate the decision-making. Furthermore, the project cost is not known until the final supply or construction contract has been awarded.

Construction contracts

Under the construction contract system the Owner enters into a contract with a contractor, who has been chosen through competitive tendering. The successful

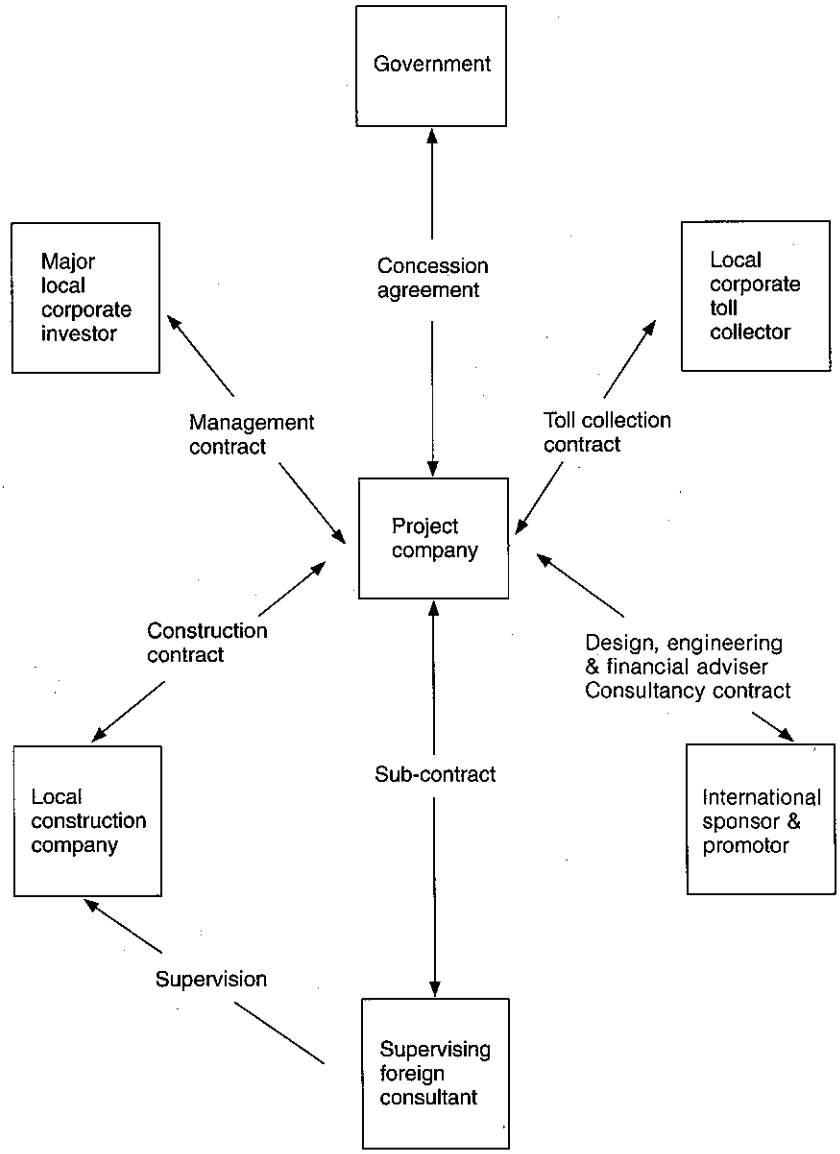


Figure 17.1 BOT contractual structure.

contractor is selected from a number of construction companies which have been invited to give binding price offers to carry out certain well-defined works. This process is often referred to as ICB (international competitive bidding) or LCB (local competitive bidding). To assist in project administration and to supervise the work of the contractor it is common for the Owner to appoint a firm of consulting engineers, often the same firm that designed the project. One advantage of the construction contract is that responsibilities between the above-mentioned three parties (the Owner, the contractor and the consulting engineers) are well defined. As the design must have been completed and detailed tender documents prepared

beforehand, the method enables fair competition between the tenderers. Competition stimulates efficiency and ascertains the lowest possible cost of the project. The owner will know his financial obligation before commencement of construction. Of particular importance for developing countries is the fact that international lending agencies as a rule will require that the construction contract method be adopted. It should be noted also that by employing a mixture of foreign experts and local engineers to jointly carry out the supervision of construction the Owner is able to upgrade his own technical staff. Drawbacks of construction contracts are that they tend to be more time-consuming than other project implementation methods, and financial conflicts between the Owner, the consulting engineers and the contractor do frequently arise.

17.2 TYPES OF CONSTRUCTION CONTRACTS

Formal agreement

For all the project execution methods discussed above – with the exception of in-house execution – the projects will be constructed under a formal agreement known as a contract.

Standard contract

Contracts are generally complex legal documents, which – if disputes arise – are liable to be challenged in arbitration courts. Most countries have therefore adopted standard contract documents, which have been tried and tested for many years under a variety of judicial systems.

Legal advice

Turnkey, BOT and project management execution methods tend to vary greatly from project to project and can therefore not be adequately covered by any standard contract. Before entering into any contract for these types of projects all parties are strongly advised to seek legal advice. Failure to do so can prove costly indeed.

FIDIC

The construction contract system, however, is well suited for the use of standard contracts, the most widely recognized of which is known as FIDIC (Federation International des Ingenieurs Conseils). Other contract systems, e.g. the British ICE, are in essence similar to FIDIC.

Traditionally, civil engineering works have been carried out under the following main categories of construction contracts:

- lump sum;
- cost plus;
- target price;
- bills of quantities (BQ).

The main features of the above types of contract are described below.

Lump sum

In the lump sum contracts the Contractor is entitled to be paid a fixed price for completing all works described in the tender. The snag is that the Contractor's obligation is taken to include all work considered incidental to the completion of the contract, whether or not such items of work are included in the contract documents. Although an experienced contractor, in most cases, might be able to foresee such incidental items, and allow for their inclusion in his tender, real problems occur when the lump sum is deemed to cover risks such as unforeseeable subsoil conditions and even errors in the contract documents, which may substantially change the scope of the work to be carried out by the Contractor. Needless

to say, contractors are generally reluctant to carry out works under this type of contract due to the high degree of risk involved. Even owners are as a rule not well served by lump sum contracts (other than for very simple and straightforward works) due to the likelihood of endless disputes following disagreements on how to interpret the various stipulations presented in the contract documents. For the above reasons, lump sum contracts are no longer used much in civil engineering works.

Cost plus

In cost plus types of contract the Contractor is paid the direct construction costs for men, machines and materials based on salary slips, equipment records, invoices and other related documentation. The only competitive element is for tenderers to quote a percentage mark-up on the direct costs, or a fixed sum, to cover overheads and profit. Such an arrangement is obviously favoured by contractors, as it involves virtually no risks and guarantees a fixed income. Owners, on the other hand, should be wary of cost plus contracts, as these offer no incentives to contractors to carry out construction in a rational and efficient manner. On the contrary, the longer the work can be stretched out, the longer will the contractor gain a secure income on his construction resources. Nowadays, cost plus contracts are only used for works where complexity and quantities are difficult to assess beforehand, e.g. bridge repairs.

Target price

The target price contract has been developed specifically to provide incentives of economy for the cost plus arrangements. To encourage contractors to be efficient, i.e. to complete a job as fast as possible, a preliminary cost is estimated, and on completion the difference between this target and the actual cost will be taken into account by calculating an adjustment – positive or negative – to the mark-up (or fixed sum) according to some sort of prearranged formula. Whereas target price is an improvement over cost plus contracts, it often suffers from lack of clarity as to how awards and penalties are related to the contractor's performance. Complaints frequently heard from contractors are that the target is wrongly estimated and that unforeseeable risks are unfairly causing losses to be incurred.

BQ

In bills of quantities contracts a project is broken down into a number of quantified construction items. Prior to construction, tenderers are requested to quote a unit price for each of these items. If the tender is accepted by the Owner, these unit prices become the basis for payment by measuring the work done item by item. The FIDIC standard contract is the system most widely used for projects valued by bills of quantities.

Over the last couple of decades FIDIC based construction contracts have rapidly been gaining ground in developing countries, and is today easily the most common method for implementation of road projects. For this reason many of the remaining sections of this chapter are dedicated to discussing project execution aspects under the FIDIC system.

17.3 THE PARTIES TO FIDIC CONTRACTS

There are three parties to FIDIC contracts:

- the Employer (Owner), who arranges for the project financing and the design of the works in addition to employing the Engineer and the Contractor;
- the Engineer, who supervises the Contractor's work;

- the Contractor, whose tender has been accepted by the Employer for the construction of the works.

Employer

After having appointed a firm or entity to act as the Engineer for the project the FIDIC condition envisages that the Employer limits his direct involvement to land acquisition and payment matters. Supervision and project administration must be left to the Engineer. In countries where the FIDIC concept is new, problems frequently occur as the Employer continues to interfere in day-to-day project activities. This leads to confusion and blurring of responsibilities, and the Engineer's position will become untenable if important matters are agreed directly between the Employer and the Contractor.

Engineer

The Engineer's duty is to see to that the project is executed in accordance with the terms of the contract between the Employer and the Contractor, and to supervise the construction to ensure that the works are built in accordance with the drawings and the specifications. It is important to note that the operation of the FIDIC system is founded on the assumption that the Engineer will play the role of a fair and impartial umpire in the interplay between the Employer and the Contractor. This concept of impartiality is not always fully recognized by Employers, who expect the Engineer to be on 'their side'. Any obvious bias on the part of the Engineer is, however, prone to backfire, as such incidents can be used by the Contractor to great effect in any subsequent arbitration proceedings.

17.4 SELECTION OF CONSULTING ENGINEER

According to FIDIC conditions the Owner (denoted the 'Employer') must appoint an 'Engineer' (normally a firm of consulting engineers) to supervise the 'Contractor's' work. When the Employer, who is often a government department, wishes to employ an Engineer for a project financed by an international lending agency, it is standard practice first to prepare a long list of consulting firms that might be considered at a later stage. These firms will have to express their interest in undertaking the assignment and submit relevant information about their background and experience (letter of interest).

Long-listing

Short-listing

Based on the information received, and having carried out an investigation into the past performance of the firms in question, a short list will be prepared containing a limited number (six or so) of the consulting engineers best suited for the job. The firms on this short list will be invited to submit proposals for the consultancy services, as defined in certain terms of reference (TOR) prepared by the Employer.

Terms of reference

Normally, separate technical and financial proposals are called for.

Technical proposal

The technical proposal will typically have the following contents:

- relevant experience of the consultancy firm;
- comments on the terms of reference;
- description of the technical approach to the services;
- staffing plan and staff experience (curricula vitae – CVs);
- support facilities, for example computers, laboratories, boring and survey equipment, office, transport, etc.

The Employer will rank the technical proposal received based on certain evaluation criteria, for example:

- firm's experience in the field of the assignment: 20 points;
- work plan and response to the terms of reference: 30 points;
- qualification of staff proposed for the assignment: 50 points.

Financial proposal

The firm of consulting engineers ranked highest on the technical considerations will be called in for negotiations, mainly on the subject of cost. The financial proposal will be the basis for these cost negotiations. If agreement cannot be reached, the firm ranked second will be called for negotiations, and so on. It is important to note that it is international practice to select consulting engineers on the strength of their technical expertise rather on cost considerations. After agreement has been reached a consultancy contract is entered into between the Employer and the Engineer.

Consultancy contract

17.5 SELECTION OF CONTRACTOR

Prequalification

The selection process for an ICB project normally starts with an invitation in the international and local press, construction journals and through embassies for prequalification of contractors. Interested contractors can respond by submitting the following information:

- past experience;
- previous clients (name and address);
- present labour force;
- present plant and equipment fleet;
- current and future commitments;
- financial strength.

Notice of tender

The number of contractors seeking prequalification can be very high. By a process of evaluating the information submitted, seeking further clarification and making one's own investigations a list is compiled of the contractors that have the requisite qualifications for the job. These successful contractors will in due course be notified when the tender documents are ready to be collected and will receive a set of documents against payment of a document fee of, for example US$500. Alternatively, the contractor may be required to pay a tender deposit, say US$5000, which will be forfeited if the tender documents are not returned.

Instructions to tenderers

Together with the tender documents the contractors will receive 'Instructions to Tenderers', which will normally have the following contents:

- instructions on how to complete the tender;
- address and time for submission of tender;
- procedures for clarification of doubts during the tender period;
- procedures on how to submit alternative tenders, if any;
- amount of security, typically amounting to 0.5% of the estimated tender sum (in the form of a bank guarantee) to be submitted with the tender; the tender bond will be forfeited in the event the tenderer does not abide by his tender, should he be awarded the contract;

Tender bond

- declaration about the tenderer's obligation to visit the site and acquaint himself with all local conditions which may affect the construction (such as access, subsoil conditions, weather, nature of work, availability of materials and local labour supply);
- information about supplementary documents available, for example geotechnical and hydrological investigations, calculations, etc., which are not part of the tender documents;
- declaration to the effect that the Employer is not bound to accept any of the tenders received, and that the cost of tendering is the responsibility of the tenderers.

Tender period

For ICB road projects the tender period is normally three months. If during this period the tenderer is in doubt about any aspect of the project or the tender documents, he must seek clarification in writing from the Engineer, or bring the matter up at the tender meeting which is arranged by the Engineer and the Employer for all tenderers.

Tender meeting

Tender opening

The tenders must be delivered in sealed envelopes to the Employer within the date and hours stated in the Instructions to Tenderers. It is common practice that all tenders are opened immediately after the deadline in the presence of those tenderers who wish to participate. For each tender the total tender sum is read out aloud together with a statement as to whether or not the tender bond is in order.

Tender evaluation

Next, the Engineer will carefully scrutinize the tenders received and prepare a tender evaluation report for the Employer. This task comprises:

- checking for arithmetical mistakes in the priced bill of quantities;
- checking whether there are any unacceptable reservations or conditions;
- evaluating alternative tenders, if any;
- evaluating foreign currency requirements;
- evaluating the work programme, construction methods and proposed plant and equipment;
- checking for an unbalanced tender (for example very high unit prices for early work items or small quantities);
- checking the degree of subcontracting and qualifications of subcontractors;
- comparing the tender sum with the Engineer's cost estimate.

Winning tender

Since a considerable period of time is likely to have elapsed since prequalification of contractors it may be a good idea, before a final recommendation is made, to have another close look at the two or three best placed tenderers, especially their financial strength and work commitments. If nothing untoward surfaces, the lowest tender is almost always selected. The only exception to this rule is if the lowest tender is much less than other bids and the Engineer's estimate. In such a case, great caution should be exercised, as experience has shown that a loss-making contract is bound to be fraught with problems from the onset of the construction.

Letter of acceptance

After the Employer has accepted the Engineer's tender evaluation recommendation, and possibly after negotiation with the winning tenderer, a letter of acceptance is issued. By this letter the contractor is advised that his tender has been accepted and that he will be called upon to sign a contract with the Employer. At the time of signing, the contractor is required to produce a bank guarantee (normally 5% of the tender sum) as security for due performance of his obligations

Performance bond

under the contract. Following the signing of the contract, the Engineer will issue an order to commence work.

Subcontractors

The Employer will only enter into a contract with the main contractor, who has been selected as described above. Subcontractors are employed by the Contractor under separate contracts which have no influence on the contract between the Employer and the Contractor. It follows that the Contractor is responsible for the conduct and performance of his subcontractors. Subcontractors shall, however, always be approved by the Engineer.

Nominated subcontractors

Subcontractors nominated by the Employer himself following special tendering, will similarly have to enter into separate contracts with the main Contractor. In such cases the main Contractor will still be the responsible party as far as the Employer is concerned. Nominated subcontractors are used in cases where the Employer wants particular parts of the work to be carried out by specialized companies, for example overhead road lighting.

17.6 FIDIC CONTRACT DOCUMENTS

The several documents, which together form a FIDIC construction contract, can logically be divided into a legal part, a financial part and a technical part.

Law

The legal part consists of the following documents:

- the signed contract;
- letter of acceptance;
- tender letter;
- performance bond;
- conditions of contract;
- addenda to the tender documents.

Economy

The financial part consists of the priced bill of quantities.

Technical part

The technical part consists of the following documents:

- drawings;
- specifications;
- reference information (frequently not part of the contract documents).

The above documents form the basis for tendering in so far as the tenderer is required to fill in the bill of quantities and complete the tender letter based on the legal and technical documents. After the contract has been signed by the Employer and the Contractor, the tender documents are referred to as the contract documents. A brief introduction to the various documents is given below.

Contact agreement

The contract agreement, which is signed by the Employer and the Contractor, simple confirms that the Employer has accepted the Contractor's tender and will pay him as stipulated in the contract. The Contractor on his part undertakes to commence and carry out the works in accordance with the contract. Finally, the documents forming part of the contract are identified.

Letter of acceptance

The letter from the Employer, informing the Contractor that his tender has been accepted, is for the sake of order made part of the contract documents.

Tender letter

The tender letter is submitted by the contractor with his tender and contains the following main points:

- an offer to carry out the works in conformance with the tender documents for a price amounting to the sum of all the items in the priced bill of quantities;
- acknowledgement of all addenda issued to the tender documents;
- a statement that the tender shall be valid and in force for a certain stipulated period of time;
- acceptance that the tender bond shall be forfeited if the Contractor should fail to execute the contract when called upon to do so.

Performance bond Declaration from a bank that it will unconditionally pay the Employer a certain specified sum of money (normally 5–10% of the tender sum) if the Contractor should fail to carry out his obligations under the contract.

Condition of contract The responsibilities of the parties to a FIDIC contract, as laid down in the conditions of contract, are set out in general terms below:

- The Employer has two principal duties, namely to give possession of site to the Contractor and to pay the Contractor his due, within certain time limits laid down in the contract.
- The Contractor is under an obligation to execute and maintain the works with due care and diligence and subject to the provisions of the contract.
- The Engineer must make decisions, issue certificates and give instructions as specified in the contract.

Addenda The contract will include clarifications and modifications to the tender documents, issued by the Engineer to all tenderers during the tender period.

Bill of quantities The bill of quantities (BQ) is a list of all items of work, which the Contractor is required to do. For each item there is a very short description and the estimated work quantity is given. The contractor has in his tender inserted unit rates and total estimated price for each individual item.

Dayworks The BQ will normally contain a list of dayworks, which is the Contractor's hourly daily rates for various types of labourers and equipment. It is normal to quote a mark-up to be paid on top of the purchase price of materials. Dayworks will only be considered in respect of minor works for which there are no other tender rates.

Schedules to BQ Attached to the bill of quantities are a number of schedules which the Contractor has filled in as part of his tender. Typically these schedules contain the following information:

- cash flow projections;
- preconstruction price adjustment indices to allow compensation for fluctuations in the cost of materials and labour;
- the Contractor's on-site organization and key personnel;
- proposed subcontractors, if any;
- lists of plant and equipment to be committed to the project;
- land requirements for temporary works, for example precasting yard and Engineer's office;
- local labour requirements by category.

Drawings Detailed drawings are essential for any civil engineering project. For a road the drawings would normally comprise plans of horizontal and vertical alignment, typical cross-sections, details of services affecting the project, road furniture, lay-out plans and detailed requirements for structures.

Specifications

The specifications are supplementary to the drawings and give particulars as to the extent of the works as well as the quality of materials and workmanship that has to be attained. This is a very comprehensive document, often running into several hundred pages.

Reference information

Certain reference information is often made available to the tenderers, for example geotechnical, climatic and hydrological conditions as well as information on materials sources. This document normally contains a disclaimer to the effect that the Contractor is himself responsible for the interpretation of the data and is under the obligation to make such independent checking as would be reasonable under the circumstances. The reason for this caution is that the majority of the larger cost claims submitted by the contractors stem from unforeseen subsoil conditions. The reference information is therefore excluded from the contract documents in many countries, even though it is made available for reference purposes.

Precedence of documents

With the great number of documents that form the construction contract it is virtually impossible to prevent inconsistencies from arising. It is important, therefore to define in the contract which documents take precedence over others in the case of conflicts. Normally the legal documents would take precedence over the financial parts, which in turn would prevail over the technical documents.

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18 Construction supervision

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18.1 INTRODUCTION

Need for supervision

The party wanting to implement a project – usually referred to as the Employer – wants to be assured that this undertaking is carried out to the required quality, within the allotted time and without any cost overruns. To look after his interests in these respects the Employer needs to make arrangements for someone to be responsible for construction supervision. Depending on the size and complexity of the project as well as the availability of skilled staff within his own ranks, the Employer may create his own supervision organization, or he may look around for a consulting firm to carry out this task. The firm or entity appointed by the Employer for the purpose of construction supervision is commonly known as the Engineer.

FIDIC contracts

The role of the Engineer will, of course, depend on the contract entered into between the Employer and the third player involved in the execution of the project, the Contractor. This chapter deals with the duties and the responsibilities of the Engineer under FIDIC-type construction contracts, which are becoming increasingly common in developing countries.

In regions where the FIDIC concept is not well known it is frequently found that whereas the Engineer's organization is well versed in all aspects of supervision, having received prior training, the Contractor is quite unaware of what will be required of him. When such things as stringent quality control measures come as a complete and unpleasant surprise, serious delays often result from the unexpected need to arrange operations to allow for rigorous testing and inspection of works and materials. To avoid such unpleasant – and expensive – occurrences, contractors would be well advised to familiarize themselves with the contents of this chapter.

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18.2 SUPERVISION ORGANIZATION

It is the duty of the Engineer to ensure that the works are built in accordance with the drawings and specifications, and that the Contractor carries out all his obligations under the construction contract.

Resident Engineer

For the purpose of day-to-day supervision of the works, the Engineer will appoint a person, denoted the Engineer's Representative or, more commonly, the Resident Engineer (RE). Under the FIDIC conditions, however, it is the Engineer who has all the important powers and authorities to act under the terms of the contract. The Resident Engineer has no authority other than what might be delegated to him in writing by the Engineer.

Delegation of powers

It is a precondition for an efficient supervision organization that the Resident Engineer is given authority to act in respect of all routine matters on the site. The Engineer is normally not present at the place of work, and it would be impractical, to say the least, to await his decision on site matters, which always tend to be urgent. For example, extra expenses would be incurred if plant and equipment have to lie idle while awaiting a decision from the Engineer, and quality problems would arise if the supervision organization is unable to stop faulty works immediately.

In international practice the Resident Engineer is normally delegated the powers needed to deal with the issues mentioned below:

- clarify discrepancies between various contract documents, warn the Contractor if the progress is too slow, suspend the progress of work when immediate action is necessary for protection and security reasons, ensure that project insurances are kept in force;
- issue further drawings and instructions as required, give reference points for the Contractor's setting out;
- approve work programmes, order daywork, take care of as-built drawings;
- preside at site meetings and issue minutes, keep records of all communication with the Contractor, deal with third parties and keep records thereof, test and approve the works and keep records;
- participate and keep records of measurements, forward monthly payment certificates to the Engineer, make periodic budget revisions;
- make recommendations to the Engineer on variation orders and time extensions, strive to prevent or at least minimize cost claims, study such claims and make recommendations to the Engineer.

Powers not delegated

More important legal and financial matters, which it is normal not to delegate to the Resident Engineer, comprise:

- major variation orders;
- time extension and cost claims, disputes and arbitration;
- certification of Contractor's default, completion and maintenance certificates;
- final payment.

RE's staff

To carry out his work the Resident Engineer needs to be supported by well-qualified staff. Apart from technical competence, such staff should have personal qualities that would enable them to work well together with the Contractor's

personnel to reduce the inherent friction between these two parties. The Resident Engineer's staff will consist of:

- engineers, with responsibility for both technical and administrative matters;
- supervisors/technicians who will be working on the site and in the laboratories;
- subprofessional staff for administration, keeping accounts, typing, filing, receiving telephone calls and the like.

Expertise

In developing countries there is a tendency to assign freshly graduated engineers and technicians to supervision duties. This should be discouraged, as technical knowledge alone will prove insufficient when the supervisor is pitted against the Contractor's seasoned foremen and superintendents in the hurly-burly of a busy construction site. It is essential, therefore, that the Resident Engineer and at least a hard core of his staff are in possession of practical road construction experience. Apart from technical know how, senior supervision staff need to be good administrators and be acquainted with accountancy and contract law.

Personal qualities

Personal qualities are equally important, as supervision engineers need to get on with people and must be able to deal with the Contractor's staff in a fair and reasonable, yet firm manner. This is not as easy as it sounds in the often charged atmosphere on a construction site. Honesty and professional integrity are essential qualities when it comes to resisting offers of monetary awards or other gifts from the Contractor.

Setting up supervision

It is important that the supervision organization is already set up and functioning at the time of commencement of work. Preparations should include a clear organization plan with lines of command and delineation of responsibilities. The number of the staff required will depend on the size of the project as well as its complexity. Too small or too large supervision organizations are both bad economy. If the supervision staff is insufficient, the chances are that work quality will suffer and substantial extra costs will be incurred as a result of successful cost claims for disruptions and delays to the Contractor's operations. A too large supervision is of course in nobody's interest.

In countries which have had little past experience with FIDIC-type construction contracts insufficiently staffed supervision organizations are all too common. This well-meant attempt to save a little bit of money will more likely than not have disastrous consequences in the form of poor quality work and large cost overruns. A rough guide for estimating the supervision staffing required for a road project in a developing country is shown in Table 18.1.

Table 18.1 Road supervision staffing requirement.

	Number of staff per US$ million annual production		
Project complexity	*Engineers*	*Technicians*	*Subprofessionals*
Simple	0.20	0.75	0.10
Average	0.25	0.90	0.20
Above average	0.32	1.10	0.25
Complex	0.40	1.30	0.30
Very complex	0.50	1.50	0.35

Example

A road is 60 km long, the tender sum is US$50 million, and the construction period is 2.5 years, i.e. annual production is 50/2.5 = US$20 million. Project complexity is rated as 'above average' (there are some quite large bridges). Number of staff required:

Engineers:	$0.32 \times 20 = 6$
Technicians:	$1.10 \times 20 = 22$
Subprofessionals:	$0.25 \times 20 = 5$

A typical supervision organization for the above road project is shown in Figure 18.1.

Facilities

In order to carry out their duties the supervision organization must be provided with such facilities as:

- permanent and mobile offices, fully equipped with telephone, telefax, radio communication, desktop computer, copying machine and draughting equipment;
- central and field laboratories, fully equipped for all standard testing of soils, pavement materials and concrete;
- survey equipment, including theodolites, automatic levelling instruments, electronic distance meters, measuring tapes, etc.;
- transport, including four-wheel drive vehicles as required.

It is common practice for the supervision organization's facilities to be supplied and maintained by the Contractor under the construction contract. After the project is completed the facilities may revert to the Contractor or may become the property of the Employer.

18.3 QUALITY CONTROL

Quality assurance is the principal reason for having a supervision organization on the site. To guarantee the quality of works it is necessary to establish close control

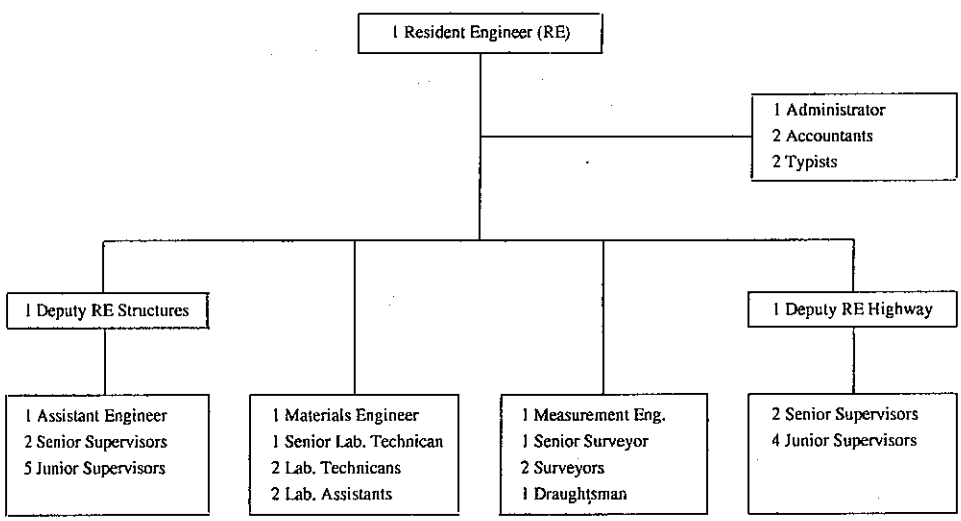


Figure 18.1 Typical road supervision organization.

with the contractor's workmanship and materials. This task can be divided into the following three groups:

Inspection
- The works are inspected visually to detect any deviation from the specified requirements. This activity is supplemented by simple *in situ* checking, for example measurement of spacing and cover of reinforcement bars, layer thickness, temperature of asphalt hot mix, concrete slump, etc.

Laboratory testing
- Materials as well as the finished product are subject to laboratory testing. For example, checking of asphalt concrete works comprises, firstly, testing of the component materials, i.e. crushed rock, sand, filler and bitumen, secondly, verification of the Marshall parameters of the hot mix and, finally, compaction control of the asphalt pavement.

Geometric control
- Geometric control is required to check compliance with specified dimensions and tolerances, for example length, width, height and camber of precast, prestressed girders and the evenness of finished surfaces such as concrete decks and asphalt pavements.

Testing facilities

Before the Contractor can start his construction activities it is necessary that the supervision organization is established and provided with testing facilities. This means that permanent and mobile laboratories must be ready, laboratory equipment must have been acquired and commissioned, test sheets and journals must have been prepared, and last but not least, testing personnel must have been employed and received the required training. The Contractor is often responsible for providing testing facilities for the Resident Engineer and his staff. It should be noted that it normally takes a long time to establish a functionable laboratory. In developing countries the test equipment will probably have to be imported, and this often causes delay. Not surprisingly, conflicts therefore frequently arise on the site, as the Contractor wishes to push ahead with earthworks and concreting at a time when the supervision organization is not ready to carry out the required quality testing.

End-product specification

There are two types of quality assurance specifications. One requires testing of the works to check that the end-product complies with the specified requirement. It may, for example, be prescribed that the compaction degree measured at any point in an earth embankment shall be not less than a given value. Where this type of specification applies the Contractor is in principle free to construct the embankment as he wishes, as long as he can attain the specified end-product requirement.

Method specification

At the other end of the scale, the emphasis is not on testing, but on the methods which may be used during construction. For the compaction of an earth embankment, for instance, this type of specification would prescribe a certain number of passes of a certain combination of rollers on a layer of earth of a certain thickness and a certain moisture content. The exact method will have to be established based on comprehensive initial trials to ascertain what combination of roller passes, etc., will meet the design criteria. It is important for the author of a specification to make up his mind as to what type of quality assurance requirements he will adopt. A mixture of end-product and method-type stipulations will easily lead to confusion during construction, unless it is made clear which requirement takes precedence.

Test methods

Apart from technical requirements the specifications will normally contain detailed descriptions of test methods, often by reference to internationally acknowl-

edged test designations, such as AASHTO (American Association of State Highway and Transportation Officials), ASTM (American Standards for Testing and Materials) and BS (British Standards). Well-defined test methods are mandatory, if disputes are to be avoided during the construction period. It is, for instance, not sufficient to simply require that a class of concrete shall have a seven-day strength of at least 30 MPa. This requirement is meaningless unless such characteristics as the test specimens' shape (cube or cylinder) and dimensions, the rate of load application and the curing conditions are defined.

Extent of control

As works shall be to the satisfaction of the Engineer under the FIDIC system, it is up to the supervision organization to determine the degree of checking to be adopted for any road project. It goes without saying that this choice will depend on the type of work at hand. Particular care should be taken for operations where there is a close relationship between the quality of the finished product and the Contractor's workmanship.

Standing supervision

Strict supervision is also necessary whenever it is difficult to rectify faulty work, and when work subsequently will be covered up. Pile driving, concrete casting and asphalt paving are examples of work items that require particular attention. In such cases there should always be an inspector from the Resident Engineer's staff present. This is referred to as standing supervision. Other work operations, which are less sensitive to the workmanship employed, or can be rectified without undue difficulty, will only require periodic inspection. Earthwork, for example, would fall into the latter category of supervision.

Test frequency

The test frequency will depend on the quality parameters which require checking. Parameters that are prone to show considerable variation, for instance composition of asphalt concrete and compaction of asphalt pavement, should be subject to running control. In such cases it is common practice to carry out a predetermined number of investigations, for example one complete extraction and Marshall test per 250 tons of asphalt concrete produced or three compaction tests per 2000 m^2 pavement laid. During the starting-up period, and whenever quality problems have been identified, the test frequency should be increased.

Quality parameters which normally remain fairly constant, e.g. the Los Angeles abrasion value of rock from one and the same source, require only occasional checking. The same goes for parameters which can be reasonably accurately checked by eye, for example the shape of crushed aggregate particles. For materials manufactured under controlled factory conditions, such as steel, cement and bitumen, it is common practice to make only a few random checks and otherwise rely on the manufacturers' certificates.

A problem every Resident Engineer will encounter is to decide on the frequency of compaction testing of road embankments. There is no hard and fast rule, as the need for testing will depend on such factors as suitability of the compaction equipment, uniformity of the soil, weather conditions and experience of the Contractor's earthwork foreman. As a rule of thumb it is suggested that each embankment layer be tested at some 50 m intervals. Near structures, where differential settlements are prone to occur, the test frequency should be considerably increased, to, say one test per 10 m^3 compacted soil.

Speed of testing

When the Contractor has completed some work which requires testing, say a layer of embankment fill, he is of course eager to carry on with construction as soon as possible. This means that the supervision organization will be under

pressure to provide the test results within the shortest possible time. If standard test methods such as sand cone are used, the results will most likely only be available the next day. It is necessary, therefore, for the Contractor to plan his operations in such a way that he can carry out work at other nearby locations, while he is awaiting the results of the testing. The emergence of nuclear density testing has eliminated the need for time-consuming laboratory testing, and the efficiency of embankment construction is greatly improved where this method is utilized for compaction control.

Evaluation of test results

The supervision organization's method of evaluating test results for the purpose of acceptance or rejection of works and materials has far-reaching consequences. For roadworks the quality control is still largely based on absolute requirements and individual spot tests. Typically, an embankment shall be compacted to at least 95% of the maximum dry density achieved by modified AASHO compaction. This means that each and every test result must be equal to or higher than 95%. Whenever a test result falls below this value, the part of the embankment represented by the test should, in principle, be rejected.

Continuous variation

The problem with the above approach is that compaction test results for earth vary continuously according to the laws of probability. This is, in fact, the case for most quality parameters pertinent to road construction. When a parameter varies continuously, the value of a random test depends on the actual distribution of the test results as well as on pure chance. Most of the test results will be near the true mean, but there will always be some results that are higher and some that are lower. Occasionally, test results will appear to be bad even though the quality is actually good. Conversely, some test results will appear to be in order, even though the work or the material represented by the test is in reality substandard.

In the light of the above the supervision organization would be justified in occasionally accepting a single failing test between many good results. It is suggested that if a particular stretch of embankment is represented by 10 compaction tests, then the whole stretch could be accepted, even if one single test was marginally below the required 95%.

However, in order not to create bad precedence, and to avoid accusations of not protecting the interests of the Employer, any acceptance of work which has formally failed, should as a rule be conditional upon further testing showing acceptable results, or instructions to the Contractor requiring some sort of remedial action. If, for instance, one compaction test result is somewhat below the specified requirement, the Resident Engineer may take another test nearby and accept the work if this test is in order. Alternatively, he may instruct the Contractor to compact the area in question by, say, another two passes of the roller.

Statistic quality control

Quality control based on absolute requirements and spot tests does not ensure a well-defined quality of the product. It is for this reason that a statistical approach to quality control has been adopted within the manufacturing sector, where works and materials are accepted or rejected based on average and standard deviation considerations. A similar approach is gradually making its entrance in the road-building industry and is today not uncommon in the quality control of concrete.

Relations with the Contractor

To maintain good relations the Resident Engineer and his staff should make it a rule not to approach the Contractor's foremen or subcontractors directly with complaints about the work. If the Project Manager is bypassed in this way he will loose face and authority, which is certainly not in the interests of the supervision

organization. The Resident Engineer should take care not to be too pedantic and rigid in his decisions. On the other hand, to be fair and reasonable does not mean that he should approve work of doubtful quality. The golden rule for all supervision staff is that quality considerations must always prevail over the Contractor's wish to speed up work and cut costs.

18.4 MEASUREMENT OF WORK

Quantities in the BQ

The quantities given in the bill of quantities (BQ) are estimates only. For most of the items it is not possible during the project preparation phase to make exact calculations of the quantities the Contractor is bound to carry out. For example, the quantity of unsuitable soil that has to be hauled to spoil can only be accurately determined during construction. It is generally considered acceptable if the bill of quantities is accurate to within ±10%. For payment purposes it is therefore necessary to measure the works actually carried out.

Methods of measurement

The contract will contain exact descriptions of how the various items of work are to be measured. These descriptions are normally found in the specifications or in a preamble to the bill of quantities. Reference is sometimes made to some country's standard methods of measurement, for example the British Civil Engineering Standard Method of Measurement, published by the Institution of Civil Engineers, London. Methods of measurement often vary from contract to contract, in particular for such items as roadway excavation (cuts and ditches), borrow excavation and haulage to embankment. Some quantities, e.g. subbase and base, are normally calculated from the theoretical dimensions shown on the drawings, which means that the Contractor is not compensated for any over-fill. It is important for tenderers to be aware of these matters, as the method of measurement may greatly influence their pricing.

Notice of measurement

Measurement of completed items of work should, according to the standard FIDIC conditions, be carried out by the Engineer, who normally will delegate this responsibility to the Resident Engineer. When the latter wishes to measure any work, he should give notice to the Contractor, who must then participate in the measurement and give all necessary assistance to the Resident Engineer. In the event the Contractor should fail to attend, the Resident Engineer's measurements should be taken as correct. It is not unusual, however, for contracts to be written so as to give the responsibility of measurement to the Contractor, in which case it will be the duty of the supervision organization to check and approve the Contractor's measurement.

Monthly measurements

Monthly measurements are made for the purpose of the Contractor's interim payment certificates. For work items in progress approximate estimates will suffice. Detailed measurements must, however be carried out as soon as any item has been completed and before it is covered up.

Non-pay items

In addition to the pay items listed in the bill of quantities the Resident Engineer would do well to measure and keep records of 'non-pay items' for which the Contractor later may present claims for extra payment. Such potential claim items comprise:

- large boulders in earth excavation;
- unforeseen settlements in embankment areas;

- slides and cave-ins;
- broken piles.

18.5 PAYMENT TO THE CONTRACTOR

Mobilization advance

It is normal international practice for the Contractor to receive an advance payment at the outset of the works to cover his mobilization costs. This advance, which is typically 10% of the contract sum, is only payable after the contract has been signed and the Contractor has provided the required security for due performance (performance bond). Security is also required in respect of the mobilization advance itself. It is important for the supervision organization to make sure that both these securities are valid and formally in order.

Monthly progress payment

During construction the Contractor is entitled to receive monthly progress payments for works carried out. The procedure is that the Contractor submits to the Resident Engineer a monthly statement showing the total value of works. The format of the Contractor's monthly statement must be approved by the supervision organization. A typical summary of the monthly statement is shown in Table 18.2. The various items are briefly commented on below.

Permanent works

The value of the permanent works must be supported by detailed calculation showing measured quantity multiplied by the unit rate for each bill item.

Variations

Under the FIDIC contract the Engineer (not the Employer) is authorized to

Table 18.2 Summary of Contractor's monthly statement.

CONTRACTOR'S MONTHLY STATEMENT SUMMARY	
FOR PERIOD ENDING..... 199..	*US$*
1. Bill No. 1: General	
2. Bill No. 2: Earthworks	
3. Bill No. 3: Pavement	
4. Bill No. 4: Piling	
5. Bill No. 5: Structures	
6. Bill No. 6: Miscellaneous	
7. SUBTOTAL to date	
8. Variations (±)	
9. Price adjustment (±)	
10. Dayworks (+)	
11. GROSS VALUE to date	
12. Retention (−)	
13. Materials on site (+)	
14. Mobilization advance (+)	
15. Repayment of mobilization advance (−)	
16. Liquidated damages (−)	
17. Interest for late payment (+)	
18. GROSS AMOUNT PAYABLE (GAP) to date	
19. GAP last statement (−)	
20. NET AMOUNT PAYABLE THIS STATEMENT	

order the Contractor to carry out extra works, omit any item of work or make changes to the design of the project. If, for example, a culvert has been overlooked during project preparation, the Engineer can order the Contractor to construct it; if the embankment soil should turn out to have a lower than expected bearing capacity, the Engineer can order an increase in the pavement thickness; and if a guardrail turns out to be superfluous, the Engineer can order that it be deleted. With a view to get a better or cheaper road the Engineer can also instruct the Contractor to use other materials than those envisaged in the contract.

Variation orders must be given in writing, except when the actual quantities merely exceed or fall short of the quantities given in the bill of quantities due to inaccurate estimates during project preparation.

For variation orders it is often necessary to establish separate bills of quantities. The value appearing in the summary of the monthly statement is the sum of all variations, each of which has been arrived at by multiplying the measured quantity of each item in the variation bill of quantities by the corresponding unit rate.

Price adjustment

It is common in international construction contracts to have a mechanism for adjustment of the Contractor's payments in the event of price fluctuations of construction materials, fuel, manpower and the like. Such adjustments may be based on documented actual costs, which is cumbersome and unwieldy, or it may be based on a formula, an example of which is shown below.

$$\text{ADJ} = K + k_B\frac{B_1}{B_0} + k_C\frac{C_1}{C_0} + k_S\frac{S_1}{S_0} + k_T\frac{T_1}{T_0} + k_F\frac{F_1}{F_0} + k_L\frac{L_1}{L_0} - 1$$

where:

ADJ	= adjustment factor for current payment certificate;
K	= fixed proportion of the contract price that is not to be adjusted for price fluctuations, e.g. overheads and profit (e.g. $K = 0.39$);
B	= price index of bitumen;
C	= price index of cement;
S	= price index of steel;
T	= price index of timber;
F	= price index of fuel;
L	= price index of labour;
0	= subscript denoting price index at the time of tender;
1	= subscript denoting current price index;
$k_B, \ldots, k_L$	= fixed proportions of the construction cost of bitumen (e.g. $k_B = 0.14$), cement (e.g. $k_C = 0.17$), etc.

The coefficients K and $k_B, \ldots, k_L$ have to be inserted by the Employer in the tender documents based on the characteristics of the particular road project. The price indices are normally published by the government in monthly statistics.

Dayworks

The Resident Engineer may order the Contractor to carry out minor work, for which there are no unit rates in the bill of quantities, as dayworks. Such orders shall be in writing, and the works will be paid according to the rates given by the Contractor in his tender for equipment and labourers. If materials are needed, these will be paid as the purchase price (verified by receipts) plus a certain mark-up (normally 30–40%) to cover overheads and profit. The Contractor must each

day submit to the supervision organization an account of the men, machines and materials utilized.

Retention money

It is normal for the Employer to withhold a certain amount of each progress payment, typically 10% of the value of the works carried out. However, these deductions are discontinued when the total amount withheld reaches a certain proportion of the contract sum, e.g. 5%. Upon due completion of the project the retention money will be returned to the Contractor.

Advance on materials

To help the Contractor's cash flow it is normal to pay an advance amounting to 75% of the value of construction materials brought to the site and not yet incorporated in the works. For the purpose of the Contractor's monthly statement a survey is carried out to determine the actual value of the materials on site at the end of the month in question. In this way account is taken of increases caused by new consignments of materials and decreases due to materials being incorporated into the works. After the advance has been paid, the materials become the property of the Employer.

Mobilization advance

The advance, paid at the outset of the construction, will appear in full in each of the Contractor's monthly statements. The mobilization advance is paid back progressively by the Contractor according to rules laid down in the contract, so that at completion of the project the whole advance has been deducted in the monthly statements.

Liquidated damages

Liquidated damages is an amount of money to be paid by the Contractor to the Employer for losses incurred by him when the Contractor fails to complete the project on time. The amount to be paid will be defined in the contract documents and is normally in the order of 0.05% of the contract sum per day the contract period is over-run. In some countries it is customary to set a ceiling on these deductions, for example 10% of the contract sum.

Interest for late payment

It is becoming increasingly common to compensate the Contractor in the event of payments being made after the time limit laid down in the contract, e.g. 45 days. This compensation would normally be tied to the country's bank lending interest rate.

Amount payable

The accumulated sum of all the above-mentioned items will give the gross amount payable to the end of the period covered by the monthly statement. The net amount payable for the current month is found by subtracting the amount paid under the last statement.

Payment certificate

The Contractor must present his monthly statements to the Resident Engineer as soon as possible after the expiry of the month in question. The supervision organization will then check the statement and issue a payment certificate within a given period of time, typically 30 days. The Resident Engineer is authorized to make corrections if there are errors or if certain works are not up to specified standard, but he is not allowed to withhold the payment certificate. After the payment certificate has been signed it will be forwarded to the Employer for payment.

Computer programs

Computer programs have been developed for the preparation of monthly statements and payment certificates. This has greatly simplified the process. As the interrelations between the various components regulating the payment due to the Contractor are defined in the program, all that is needed each month is to adjust the quantities of works carried out and to feed in such variables as price adjustment indices and quantities of materials on site. The more advanced programs are even capable of utilizing these data to plot up-to-date progress charts.

Final payment

Upon formal completion of the construction, which is marked by the issue of a completion certificate, the time has come to prepare the final payment certificate. This document is a full and substantiated account of the project costs and will show the exact amount of money due to the Contractor. Half of the retention money will normally be released at this point in time.

Contract completion

After a guarantee period – normally one year – there is a final inspection, and the maintenance certificate is issued, if there are no shortcomings that can be blamed on the Contractor. The last half of the retention money and the performance bond can now be released, after which the Contractor has no further obligation under his contract with the Employer.

18.6 PROGRESS CONTROL

Progress tool

The FIDIC contract conditions have stringent requirements about the Contractor's obligation to produce and constantly update work programmes. The planning of the works is the Contractor's responsibility, but the work programmes need to be approved by the Resident Engineer. All the three parties to the construction contract have a vested interest in programmes, as these are essential tools to control the timely completion of the project with due regard to quality cost. They also form the basis for the Employer's cash flow requirements and planning of the supervision activities. Finally, work programmes are necessary for co-ordination with utility owners and other contractors.

Method statement

The Contractor must submit a work programme within a certain period of time after the Engineer's order to commence work. Upon request he is also required to submit detailed statements of the methods he proposes to adopt for key items of work, complete with identification of plant and equipment. It is the duty of the Resident Engineer to approve or disapprove the work programme. If he is in doubt he should not hesitate to request the Contractor to submit further information or clarification. The importance of a detailed and realistic work programme at the outset of the works cannot be over-emphasized.

Programme appraisal

In the assessment of the work programme the Resident Engineer will have to consider if there is a reasonable relationship between the time allocated for various work operations and the proposed input of machines and labour. Have the qualifications of local labour been taken into account? Are equipment output figures realistic? Has due consideration been given to local holidays and climatic conditions, for example rainy seasons, when work efficiency will be reduced? Has sufficient time been allowed for mobilization? Have all known obstructions and constraints such as access to site problems, sectional handing over of the site, the need to construct temporary bridges, late removal of buildings, etc., been incorporated into the work programme? If the answer is no to any of the above questions the work programme should be returned for amendment. Upon being made aware of any problems, the Contractor will either have to improve the efficiency of his operations or he will have to mobilize more equipment and labour. Even approved programmes will have to be adjusted by the Contractor, if the Resident Engineer deems that work is falling behind schedule.

Types of programme

Work programmes can be prepared in a number of ways. Common to all types is that the programme should be easy to read and easy to monitor. Simple bar charts, where each activity is drawn as a bar on a time-scale, are more than adequate for

most road projects. As bar charts tend to become difficult to interpret if too many activities are shown, it is a good idea to work out supporting programmes for each of the more important operations. A shortcoming of the bar chart is that it is not very suited to show dependencies between work items, which means that it is difficult to assess the effects of localized delays. For larger and more complex work, for example bridges, it may therefore be desirable to supplement the bar chart with network planning and critical path analyses.

Progress monitoring

It is a sad fact that many contractors, especially from developing countries, do not pay sufficient attention to work programmes. These plans are of little or no value if they are used as wall decorations rather than as a tool to control work progress. For a programme to serve its purpose the progress needs to be constantly monitored, which means that actual achievement is compared with planned output. When work is falling behind schedule, it is important to determine the reasons as soon as possible. Corrective measures can normally be introduced at an early stage to bring the progress back on schedule. However, it is often seen that when the Resident Engineer requires the work programme to be adjusted due to delays, all that happens are some cosmetic changes to the programme itself, while nothing is done about the reasons for the delay. This amounts to attacking the symptoms rather than the disease..

Example

An example of totally unrealistic planning is shown in Figure 18.2. After four years of hopelessly inadequate progress, i.e. twice the original contract period, the Contractor in question became bankrupt and had to abandon the works with only 40% achievement to show for his efforts.

One main advantages of the FIDIC contract execution method is to get projects completed within a fixed period of time. All too often, however, large delays are incurred. The best way of preventing such delays is for the supervision organization to adhere closely to the FIDIC stipulations about work programmes and do their best to get the Contractor to take corrective measures in time. To achieve this the Resident Engineer and his staff should not hesitate to give advice, but they should not interfere directly in the Contractor's operations. To do so would mean to take on responsibility and may easily result in subsequent cost claims.

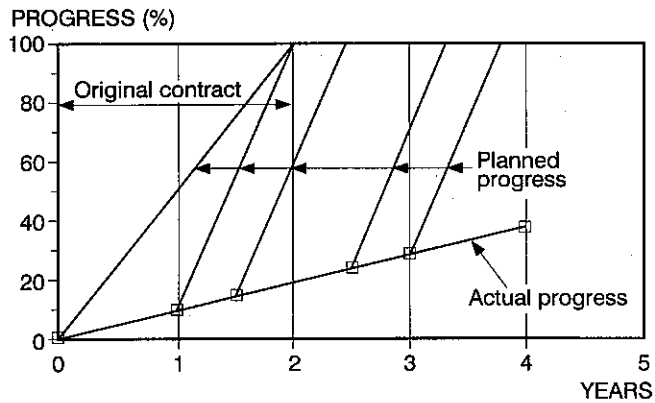


Figure 18.2 Example of unrealistic planning.

18.7 EXTENSION OF TIME

Need for time extension

If delays have been incurred, which are likely to cause delayed completion of the project, it will become a priority issue for the Contractor to seek extension of time for completion. This is necessary to reduce or eliminate the amount he will otherwise have to pay as liquidated damages by not completing on time.

Time extension criteria

Under the FIDIC contract, extension of time is justified if, for example, delays have been incurred to the Contractor's operation due to:

- late availability of the site;
- late issue of drawings and instructions;
- suspension of work;
- additionally inclement weather;
- *force majeure* situations.

To be awarded a time extension two fundamental conditions must be met. Firstly, the delay must not be due to any fault of the Contractor. Secondly, the delay must be such that it will directly influence the overall completion of the project, i.e. it must fall on the critical path.

Case history

An example of how to deal with a time extension claim is given below. The Contractor claimed that earthwork operations within the road reserve had been stopped by landowners on two occasions. In the first incident works were held up for 45 days while the Employer sorted out matters. In the second incident works were held up for 70 days. In consequence, the Contractor requested 45 + 70 = 115 days time extension.

The Engineer agreed that this claim for time extension was admissible under the FIDIC contract, and that the incidents were not the fault of the Contractor. It was furthermore agreed that there was a justifiable delay of 45 days for the first incident, as records confirmed that the team in question had been idling for the said period of time in expectation that the problem would be expeditiously resolved. The second incident, however, was different inasmuch as the Resident Engineer had instructed the Contractor to move his team to another nearby location one day after the work had been stopped. The Contractor had complied with order and was able to return and complete the earthwork after the problems of the second incident had been settled.

With regard to quantification of the time extension the Engineer noted that as the Contractor had four earthwork teams on the site, the effect on the overall production of one team idling was only 25%. Extension of time was awarded as follows:

Incident 1:	Lost time		= 45 days
Incident 2:	Lost time	1 day	
	Removal of team	3 days	
	Re-entry	3 days	= 7 days
Total			= 52 days
Real delay 25% of 52 days			= 13 days

Procedures

A request for extension of time should be made by the Contractor as soon as possible after the delay has been incurred. The supervision organization is also under an obligation to act fast, since the Contractor will not be able to plan his

operation properly as long as he does not know whether or not an extension of time will be granted. When evaluating a request for time extension the Resident Engineer is, as usual, under an obligation to be fair and reasonable. On the one hand he has to consider that to give additional time is likely to mean increased project costs due to lower or no liquidated damages as well as possible consequential cost claims from the Contractor. On the other hand, rejection of justified requests may end up costing more, if taken to arbitration (arbitration will be discussed in the next section). Extension of time should always be dealt with by the Engineer, as knowledge of contract law is essential in handling these matters correctly.

18.8 COST CLAIMS

Claim-prone contractors

Almost every road construction contract nowadays is faced with claims for extra payment. Not infrequently the aggregate value of the claims equals or even exceeds the original contract sum. Certain international contractors are deliberately tendering low to secure the contract. They then start to prepare cost claims from day one with the help of claims specialists, frequently lawyers. It is extremely important, therefore, that the Resident Engineer and his staff are conscious of claims from the outset of the project and make every effort to prevent or at least minimize extra costs. On the one hand, some Employers regard any request for extra payment as an insult and expect the supervision organization to reject all claims out of hand. This is improper, as there are certainly many claims which are genuine and deal with matters that should be the risk of the Employer under the FIDIC conditions.

Types of claims

The great majority of cost claims fall into one of the following three categories:

- the Contractor has encountered conditions or obstructions that could not have reasonably been foreseen at the time of tender;
- extra or changed works have been paid by too low unit rates fixed by the supervision organization;
- there have been delays and disruptions to work operations for reasons beyond the control of the Contractor.

Unforeseeable conditions

Unforeseeable conditions and obstructions often refer to matters which are hidden in the ground, such as problematic subsoil, poor-quality borrow-sources, water pipes and the like. The basic rule is that the Contractor is only entitled to recompense for any difficulties encountered if it can be established that even an experienced Contractor could not reasonably have foreseen what happened. It should be noted that the FIDIC conditions require tenderers to make all such investigations as are necessary to prepare a realistic tender, including subsoil conditions. If the Contractor has failed to do this, he will himself be held responsible for the consequences.

For this type of claim the Contractor will only be able to recover his actual costs; there is no entitlement to profit.

Variation claims

As previously mentioned, additional or changed work should be paid by the unit rates in the bill of quantities, if these are considered applicable by the supervision organization. In the event that existing unit rates are not applicable, or when no

unit rates exist in the bill, the FIDIC conditions require that adjusted or new unit rates be agreed with the Contractor. If it is not possible to reach agreement on what is fair and reasonable, the supervision organization has the right to fix the rates unilaterally. In doing so, however, great caution should be exercised, as disagreements about fixed rates is one of the most common causes of dispute under international construction contracts. There are numerous cases on record where arbitrations have resulted in very big awards to the Contractor, and this has often come as a great surprise to the Employer and the Engineer. Therefore, variations should not be ordered without prior serious consideration of the possible implications.

For variation-type cost claims the Contractor is entitled to be paid cost plus profit.

Delay claims

Claims for delays and disruptions to the Contractor's operation are in principle admissible in the following cases:

- the site has not been made available on time;
- drawings have been issued too late;
- the progress of the works has been formally suspended by the supervision organization;
- there have been errors and discrepancies in the contract documents;
- the survey points handed over to the Contractor have been in error;
- the Contractor has been ordered to make excavations to uncover and test work which subsequently proved to be in order.

To qualify for payment the Contractor has to prove that it was not possible for him to utilize idling equipment and labourers, for example by moving the team in question to alternative work sites. This is important, as it is the obligation of the Contractor to make all possible efforts to minimize the effects of any obstruction to his work. Furthermore, it is the Contractor's duty to document all his claimed expenses.

For most delay and disruption claims the Contractor is entitled to be paid overheads, but not profit.

Case history

The following shows the importance of evaluating claims in strict accordance with the conditions of contract. The two examples given are on the face of it quite similar, both dealing with too-late issue of drawings, but the conclusions are nevertheless totally different.

Example 1

One culvert had inadvertently not been shown on the contract drawings. When the Resident Engineer discovered this omission and ordered a variation, the culvert works were nearly completed. The Contractor immediately ordered the additional culvert pipes, but one work team, that otherwise would have been demobilized, had to remain idle for three weeks waiting for the pipes to arrive at the site. The Contractor claimed that he should be compensated for the additional costs incurred. The Engineer accepted this claim as (1) it was admissible under the contract, (2) there was no alternative work for the team in question and (3) the Contractor could not reasonably have foreseen the need for this culvert.

Example 2

One pipe culvert was shown on the drawings, but the required dimensions had inadvertently been left out. As for Example 1, culvert works were nearly completed when the Resident Engineer discovered this error and provided the Contractor with the correct dimensions. As before, one culvert team was forced to be idle for

three weeks, till the pipes of the specified dimension arrived. The Engineer rejected this claim on the ground that it was the Contractor's contractual obligation to study the drawings at the time of tender. If he had done so, he should have discovered the error and could have sought clarification. It was held by the Engineer, therefore, that the Contractor was himself responsible for the idling of the culvert team.

Misconceptions

The points below are intended to correct commonly held misconceptions about cost claims:

- The fact that a time extension has been granted does not automatically justify additional payment to the Contractor.
- Exceptional weather may entitle the Contractor to time extension, but not to extra payment.
- Whether or not the Contractor has lost money is irrelevant and cannot form the basis of a cost claim.
- Any misdeeds of a subcontractor are the responsibility of the Contractor.
- As long as the Resident Engineer and his staff act within the framework of the contract, there is no such thing as 'too strict supervision'.

Claims procedures

When raising a cost claim the Contractor has to comply with strictly formal procedures. Firstly, it is important that written notice is given forthwith about his intention to claim extra payment. Although the claim itself may be submitted later, it will not be disqualified if the notice requirement has been complied with. Secondly, to be valid a claim must be raised under a specific clause (or clauses) in the conditions of contract. Finally, the burden of proof is on the Contractor, and he needs to provide all documentation to substantiate the additional costs claimed.

Umpire

It is normal for the Engineer to retain the authority to deal with cost claims. It is the intention of the FIDIC conditions that the Engineer shall act as an independent umpire (mediator) and make fair and unbiased decisions, even if such decisions might be unfavourable to himself or the Employer.

Arbitration

If the Contractor does not accept the Engineer's decision, he shall inform the Engineer accordingly in writing. If such a dispute cannot be resolved through negotiations, it will be referred to arbitration, where it will be settled by one or more arbitrators, whose decision is final and binding. In an arbitration it is of crucial importance that the supervision organization is able to present written records in respect of instructions given to the Contractor, observations, tests, progress of work, the weather, equipment on site, personnel, material supply, errors done by the Contractor, etc. The Engineer will be in default of his obligations under the contract if he fails to produce such crucial evidence.

18.9 DEFAULT OF CONTRACTOR

Expulsion of Contractor

According to the FIDIC conditions the Employer can expel the Contractor from the site, if the Engineer certifies that the Contractor has:

- not commenced the work or has abandoned the site before completion of the works; or
- assigned the works or parts thereof to others without permission; or

- failed to correct faulty work which has been rejected by the supervision organization; or
- become bankrupt; or
- flagrantly and persistently failed to fulfil his obligations under the contract.

Completion of works

In the event the Employer expels the Contractor the supervision organization will ascertain the value of all completed and started work as well as the value of all equipment, plant, materials, scaffoldings and the like. The Employer can either complete the project himself, or he can select a new contractor to carry out the remaining works. In either case, at the discretion of the Employer, the equipment, materials, etc., of the original Contractor can be sold or made use of in completing the project. The fact that the Contractor has been expelled does not mean that his contract with the Employer has been rescinded, and he is still liable for the costs of completing the works.

Settlement of account

The account will be settled after the project has been completed by others based on the following principles. Firstly, the supervision organization will determine the payment that would have been due to the original Contractor, if he had completed the works. From this amount is deducted the actual cost of completing the project as well as liquidated damages and any additional costs incurred by the Employer as a result of the default. In consequence, the original Contractor will inevitably end up owing the Employer a large sum of money. To recover this debt, at least in part, the Employer can confiscate the performance bond as well as any proceeds from the sale of the equipment, etc. If, as would normally be the case, there is still a debt due from the Contractor, the Employer's only option is to try to recover this through civil litigation.

18.10 SUPERVISION PROCEDURES

Meetings

Meetings are an essential tool in project administration. To be successful meetings need to be well prepared, business-like and conducted in accordance with a fixed agenda. It is normal for the Resident Engineer to act as chairman. Clear and concise minutes of meeting need to be taken for subsequent circulation and approval of all participants.

First site meeting

Prior to commencement of work it is normal to hold a site meeting with participation of the Engineer, the Resident Engineer and other supervision staff on the one side and the Contractor's Project Manager (Site Agent), other representatives and subcontractors on the other side. It may be a good idea to invite the Employer to this important first meeting, which is held for the following main purpose:

- to acquaint the parties with each other, including an introduction to the Contractor's and the Engineer's organizations;
- to clarify which powers and authorities have been delegated to the Resident Engineer;
- to agree administrative routines, including lines of communication and organization of day-to-day activities;
- to review the Contractor's work programme and establish how far advanced his mobilization is;

- to verify that contract securities and insurances have been brought in order;
- to agree on the sectional handing over of the site, if relevant;
- to explain all pertinent issues to the Contractor, including relations with local property owners, utility authorities and the like.

Generally speaking, the supervision organization will at the first site meeting seek to clarify and settle any doubts the Contractor might have about efficient future co-operation.

Routine site meetings

During the construction period it is good practice to hold regular site meetings for the purpose of recording events, discussing problems and taking decisions. The frequency of these meetings varies, but monthly meetings is the accepted norm on many internationally tendered road projects. Main participants are the Resident Engineer and the Contractor's Project Manager, both supported by key technical and administrative staff. The agenda, which is circulated prior to the meeting, will often comprise the following topics:

- comments on the last site meeting;
- progress last month;
- planned progress;
- men, machines and materials;
- technical matters;
- financial matters;
- administrative matters;
- miscellaneous;
- the time for the next site meeting.

Records

It is a principal duty of the Resident Engineer and his staff to keep records of whatever transpires on the project. These records will form the basis for approval of works as well as payments to the Contractor. Furthermore, site records are indispensable in the event of disagreements, for example in respect of extra or changed work, time extensions and claims for additional payment.

Supervision records may broadly be divided into the following four categories:

- historical records, i.e. work programmes and monitoring data, Resident Engineer's diary and daily inspection records;
- quality records, i.e. test results, survey control, etc.;
- quantity records, i.e. measurements for payment, monthly statements, payment certificates and variation orders;
- 'as built' records, i.e. drawings and descriptions of all completed parts of the project.

Correspondence to and from the Contractor, the Employer and various other parties form part of the Resident Engineer's records. Because of the sheer volume of paperwork involved it is imperative that an adequate filing system is devised and adhered to from the outset of the construction period.

Progress reports

The supervision organization is normally required to prepare monthly progress reports for distribution to the Employer, the international financing agency and other parties with an interest in the project. Typically, this report will have the following contents:

- a brief description of the original project and any significant changes;

- key contract data, including any approved time extension;
- information about approved subcontracts;
- progress of work as compared to the approved work programme, identification of matters that may cause delays and action taken to bring the project back on schedule;
- physical considerations, including data on weather, construction resources and testing as well as quality problems;
- financial considerations, including last month's payment certificate, schedule of variation orders, cost claims approved and pending, and updated cash flow projections;
- supervision organization, performance and problems encountered.

Any Resident Engineer worth his salt will take pride in preparing informative and lucid monthly reports. The use of progress photographs is a good idea, as it is well known that one photograph can tell more than many words.

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19 Labour-based construction

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19.1 INTRODUCTION

Definitions

Construction technology covers a spectrum of construction methods using various mixes of labour and equipment. Construction methods are characterized as equipment-based or labour-based depending on the principal source of motive power used. Equipment-based methods are also termed capital-intensive methods as they are heavily dependent on foreign exchange for import of equipment, spare parts, and fuel. Labour-based methods are often called employment-intensive or labour-intensive because they contribute to employment creation. Some specialists prefer to talk about appropriate technology, i.e. the most suitable mixture of labourers and equipment.

Present practice

Equipment-based road construction methods developed in the industrialized countries are used extensively also in the developing countries. There are several reasons for this:

- Politicians, planners and engineers are strongly influenced by the technology used in the industrialized countries and there is a very effective lobby and sales pressure from heavy-equipment manufacturers.
- Most road contractors are used to equipment-based working methods.
- Heavy equipment is often supplied at no apparent cost to the user ministry as part of foreign funded aid packages. Also, tax exemptions distort the real cost of machine-based methods and therefore influence the choice of technology towards the use of heavy equipment.
- Local sustainability of road projects is rarely considered when projects are designed.
- Employment creation is seldom taken into account when decisions are made on project implementation.
- Lending criteria of financial institutions generally favour large-scale programmes which have higher disbursement levels than projects which are executed with local resources.

Highway and Traffic Engineering in Developing Countries Edited by Bent Thagesen. Published in 1995 by E&FN Spon, London. ISBN 0 419 20530 6

Present trend

During recent years an increasing proportion of development plans in the Third World have emphasized the need to promote the use of local resources for infrastructure programmes. Both governments and financing agencies treat this issue much more seriously than they did some years ago. Two circumstances have caused this change of attitude.

Lack of foreign exchange

Particularly in Africa and South America, increasing debt-servicing obligations have had crippling effects on national economies, forcing countries to adopt structural policies with repercussions on most sectoral activities. Restrictions on importation, often accompanied by frequent devaluations of the local currencies have resulted in foreign goods becoming extremely expensive or outright unobtainable. The effect on operations in the road sector with its traditional dependence on foreign equipment, skills and goods has been devastating. In most developing countries transport constraints are very common due to the poor state of the road network.

Unemployment

In most developing countries, annual population growth figures are in the order of 2–3%. At the same time employment opportunities particularly in the agricultural sector are on the decline. The combined effect of these tendencies is that unemployment or under-employment is rising in a large number of countries.

ILO

The transport sector has traditionally received a large share of public investment, mainly because transport is an indispensable element in the successful implementation of development programmes in most sectors. Equally, as noted, most of this investment has traditionally been spent in a capital-intensive manner. It is no coincidence therefore that in the early 1980s transport and infrastructure were singled out by the International Labour Office (ILO) in Geneva as the most important sectors in which practical employment-oriented approaches should be tested and implemented. The ILO has subsequently initiated and assisted in a large programme of labour-based civil works projects in more than 30 developing countries. The ILO-assisted projects particularly in the road sector have led to large-scale locally executed and financed labour-based road programmes in Africa and Asia.

19.2 CHOICE OF TECHOLOGY

Whether, in a particular country, labour-based road construction techniques will be viable or not, depends upon several factors (ref. 1). The most important of these are:

- government attitude;
- economic level;
- comparative costs;
- establishment of appropriate administrative and financial procedures;
- establishment of relevant management and technical training;
- labour availability;
- labour attitude;
- type and location of projects.

Government attitude

The government and the administration at different levels must have a positive attitude towards assessing alternative technologies which are not presently used in

the country. Frequently, preconceived ideas and attitudes, particularly of government engineers, militate against the introduction of labour-based road construction methods.

Economic level

In a free market situation, the level of equipment intensity or labour intensity of a particular activity should be governed by the prices given to the various factors of production. When screening countries with a view to identifying those with the best potential for labour-based techniques, the general economic level would give some indication of the type of technology that is appropriate. One factor, albeit somewhat coarse, which can be used as a measure, is the GNP per capita.

Comparative costs

In choosing the most appropriate technology for a road construction project it is necessary to make cost comparisons. The estimates should analyse the costs of alternative approaches taking into account costs of labour, staff, material, tools and

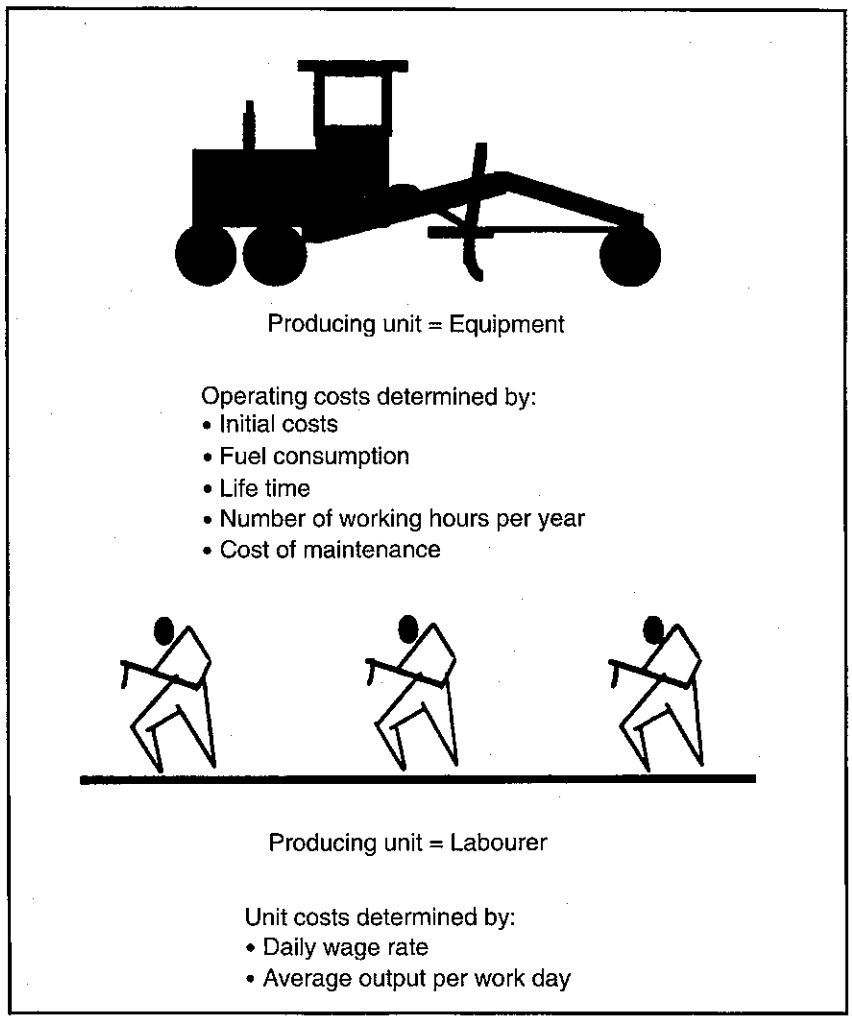


Figure 19.1 Comparison of cost between equipment and labour (copyright International Labour Office, ref. 2).

equipment, camps, offices, current expenses, and overheads. Figure 19.1 shows the most important factors to be considered when making cost comparisons. The cost of mobilization, management and planning staff at headquarters, method study personnel and training facilities should not be forgotten.

Wage rate

An important factor in the assessment of whether labour-based techniques are economically appropriate is the daily wage rate. The agricultural wage rate is a rough indicator which can be used as a factor in establishing wage rates for road works in rural areas. Research has indicated that at less than US$5 per day (price level 1984), labour-based methods should be seriously considered. In 1984 this implied that labour-based techniques were potentially viable in 56 countries (ref. 3). In the intervening years many countries have seen their economies retreat and still fall within this wage rate criteria (ref. 12).

Productivity

When calculating the cost of labour and equipment, the productivity of labour and equipment needs to be known. Labour productivity data may be taken from international sources if no systematically organized labour-based work has been done recently in the country. Such data should however be used only for tentative planning and it should be the aim to establish local data as soon as possible. Daily productivity can be assessed relatively quickly through work studies. However, only experience over a long time can give the frequency of work disruptions and hence the long-term productivity.

Productivity data on machines may be taken from manufacturers' handbooks. Allowance should be made for the fact that data provided by the manufacturer tend to be optimistic and reflect the conditions in the land of manufacture, normally an industrialized country. When assessing the long-term productivity of equipment, considerations should be given to low utilization and idle time caused by insufficient planning, breakdowns and shortage of fuel and spare parts. Figure 19.2 shows the factors that influence the productivity of labour and equipment.

Administrative and financial procedures

Output and productivity levels of labour-based projects are heavily influenced by the degree of confidence of small contractors and/or workers in their employer. Successful labour-based projects, therefore, *must* have good administrative and financial procedures which are relevant to working with large numbers of local workers.

Management and training

The difficulties in organizing, administering and controlling large labour forces are many and require special skills. Reasonable production and productivity levels can only be obtained by appropriately trained management and technical staff.

Shadow prices

For project evaluation, 'shadow' prices could be used rather than market prices. Shadow prices are a reflection of the foregone opportunity of using the resource in question for other purposes. This usually means that, because of the socio-economic advantages of using local resources (employment, skills development, local spin-offs for the rural economy) labour-based technologies could be priced at a level lower than their actual financial costs. On the other hand, the disadvantages of using foreign skills and resources (use of scarce foreign exchange, difficult sustainability) would lead to a higher pricing than the actual costs. Nevertheless, the decision to adopt shadow prices instead of market prices is one to be taken by the highest authorities in the country, inasmuch as it is partly a political decision.

Labour availability

The availability of labour, in sufficient numbers, is a prerequisite for any labour-based construction programme. Normally, the level of unemployment or under-employment is used as an indicator of labour availability. Unfortunately, these

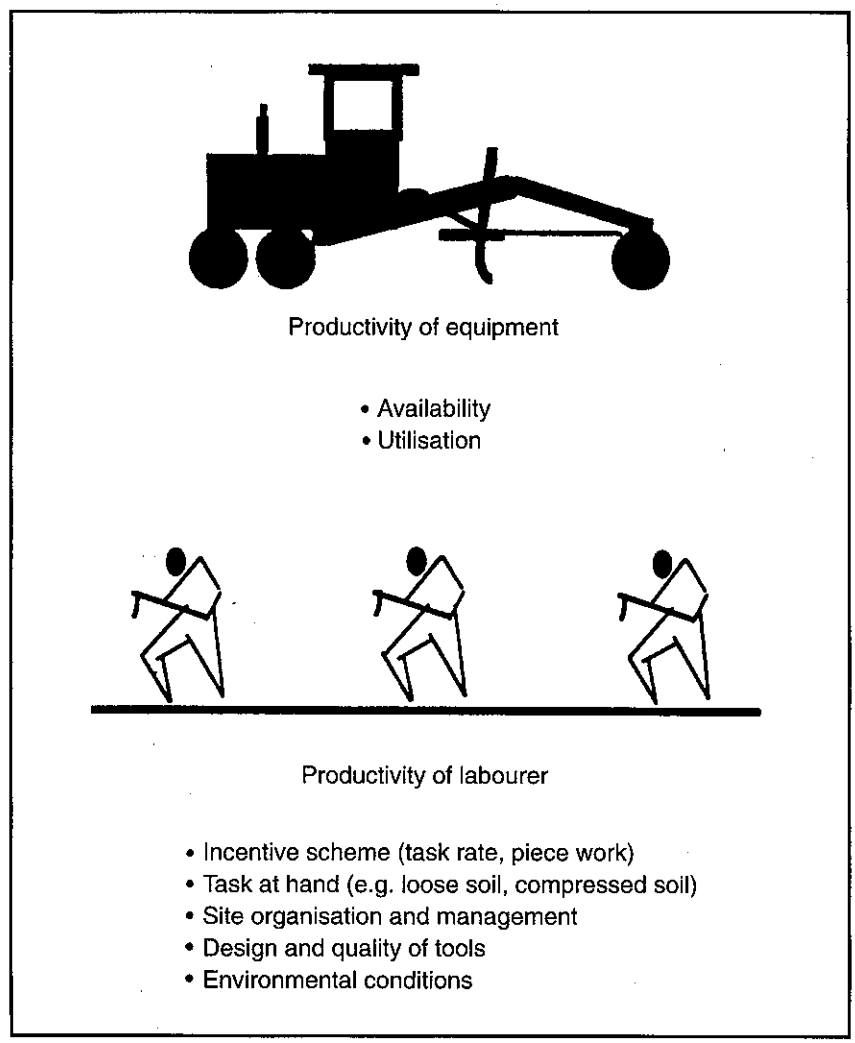


Figure 19.2 Comparison of productivity between equipment and labour (copyright International Labour Office, ref. 2).

figures are difficult to obtain, in particular in relation to the non-wage economy in the rural areas. The population density can be used as a rough proxy for labour availability, but these figures must be treated with care as a low figure does not necessarily mean that labour is not available in particular regions. In general, the ILO suggests that 25 persons per km^2 is a figure above which labour supply would not normally be a problem.

Labour attitude

The attitudes of the available labour towards the work and local customs are other factors which need consideration. Earthwork is a predominant activity in most road construction projects. In certain countries earthwork activities rank below agricultural work in the hierarchy of employment. In other countries it will not be acceptable for women to carry out certain or all road works. Therefore, it is

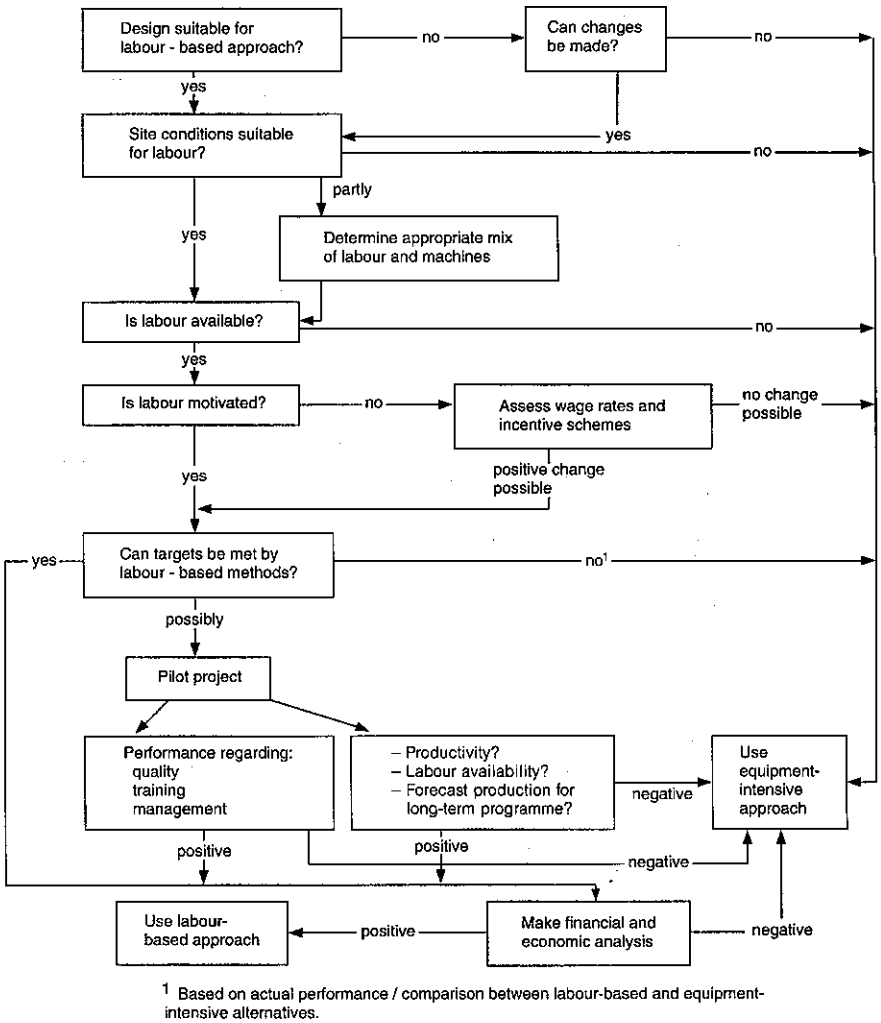


Figure 19.3 Decision-making relating to the choice of technology (copyright International Labour Office, ref. 2).

necessary to ascertain the willingness of available labour to accept employment on road works.

Project type

The type of project and its location also play an important role in the choice of technology. Labour-based methods are best suited for construction programmes comprising a large number of small, technically simple, and geographically dispersed road projects. Construction involving the movement of large quantities of materials over large distances, or substantial production within a short period of time, is best carried out by mechanical equipment. Paving with premixed asphalt requires equipment if high quality is required. For gravel road construction, all or nearly all activities can be done by labour. Also, for periodic and routine maintenance works labour-based methods are well suited; see Chapter 20.

In general, the steps to be taken in choosing technologies for unpaved road projects in developing countries are indicated in Figure 19.3.

19.3 TECHNICAL ISSUES

Design

Labour-based methods require a somewhat different approach to the design of roads from that when equipment-intensive means are used. Attention should also be paid to the working methods of the labourers.

It is necessary to adapt a road cross-section which is appropriate to the technique used. By the nature of its blade a grader can only produce triangular or straight cuts. Curved or trapezoidal sections, which are usually more stable, can however be produced by labour. The shape of the grader blade dictates a shallow V-shaped ditch. A ditch excavated by shovel will need to have a flat bottom and can be made with steeper sides.

Also, the design specifications in contract documents for roadworks are generally rigid in terms of compaction standards, thickness of base and subbase and quality of materials. A less rigid approach where thicker layers of lesser-quality material compacted to slightly lower compaction rates may not only allow for the use of different compaction methods, but may also minimize the haul distance.

Earthworks

For labour-based construction the earthwork in the longitudinal direction should be minimized as much as possible. High design speeds may not always be required for rural roads. A reduction in the design speed would allow the road alignment to follow the terrain contours more easily, reducing the earthworks.

It has proven to be feasible and economic to execute even large earthworks by labour, provided that there is not a lot of hard rock, and provided that the hauling distance is minimized. The hauling distance can be reduced by moving soil sideways rather than balancing cuts and fills along the length of the road, which is usually the case when heavy equipment is used.

Compaction

Labour-based road construction requires light compaction equipment. Large mechanical compaction equipment is difficult to move between the many sites being worked by labour crews on large construction projects, and is usually under-utilized on smaller projects.

In some cases, it may be sufficient to resort to indirect or natural compaction, i.e. compaction resulting from the action of the weather and traffic on an embankment over time. The main disadvantage of indirect compaction is the level of erosion and deformation during and immediately following the construction period. Extra workdays are required to reshape the road one or more times before the required compaction standard has been achieved. In Mexico, loaded trucks or tractors have been successfully used to achieve initial compaction. Kenya recommends compaction of embankments by loaded trucks followed by compaction with pedestrian controlled vibratory compactors.

Pavements

A number of road pavements are suitable for labour-based methods (ref. 4).

Gravel

Natural gravels are the most economical to use as they can be laid without any double handling. It is easier for labourers to lay fine gravels with a good surface finish than gravels containing large stones.

It is not feasible by labour-methods to produce the same high standard of surface finish as that produced by a skilled grader driver, but spreading gangs can



Figure 19.4 Hand-mixing hot asphalt in India.

build up considerable skill if their work is well supervised and frequently checked with string lines.

Soil stabilization

Hand mixing of lime-stabilized soil is used for example in China. However, hand mixing is inefficient for pulverizing clayey materials; lime dust may create a health problems for the labourers. Hand mixing of cement-stabilized soil is not recommended as cement reacts and sets too quickly.

Surface dressing

Surface dressing may be carried out by labour-based methods and the quality of work will be almost as good as by mechanized methods if the works are properly managed. Material specifications are the same as for mechanized surface dressing and described in Chapter 15.

Premixed asphalt

Hot premixed asphalt is sometimes produced by hand (Figure 19.4). However, it is difficult to create satisfactory working conditions for the labourers when the bitumen and the aggregate are heated over an open fire and it is not possible to obtain the same quality of mix and the same standard of surface as that produced by mechanized mixing plants and asphalt pavers. However, hot-mixed asphalt, mixed and spread by hand, give good service on thousands of kilometres of roads throughout India and China. In many other countries this type of asphalt is widely used for repair works.

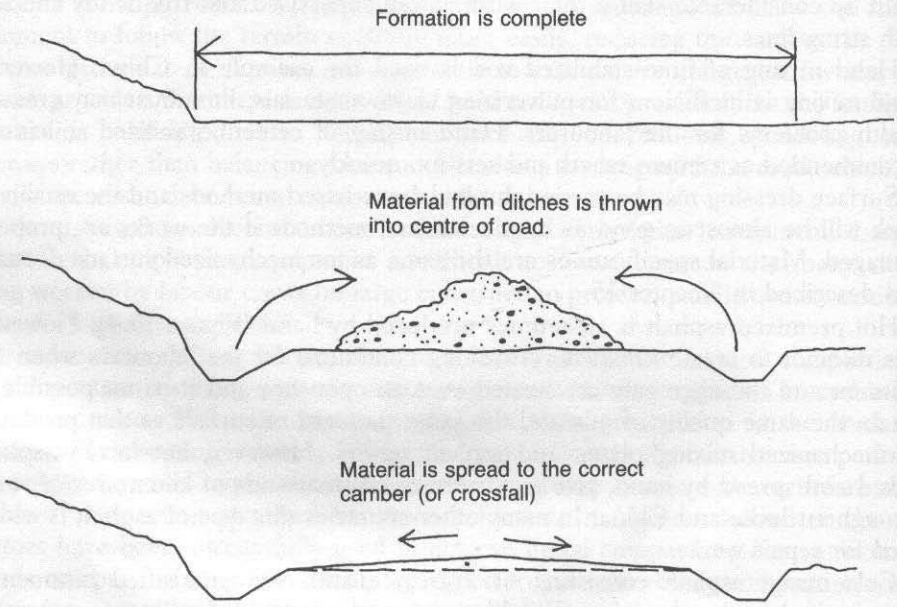
Cold-mixed asphalt consisting of aggregate and cold emulsified bitumen is better suited for hand mixing. Troublesome smoke is avoided and correct proportioning is easier. The use of cold-mixed asphalt should be encouraged.

Penetration macadam

Construction of penetration macadam, i.e. asphalt macadam made *in situ*, is well suited for labour-intensive methods. Penetration macadam consist of two or three layers of progressively smaller, angular stones. Each layer is compacted by rolling, after which it is sprayed with hot bitumen, cutback bitumen or emulsified bitumen. Penetration macadam is used extensively in Indonesia.



Figure 19.5 Stone cutting in China.



Surface drainage prevents water from damaging the road by leading it off the road quickly. This is done by shaping the road so that the water can flow freely into the side drains.

The slope from either side of the centre-line to the sides is called the **camber**. This kind of sloping is used in open terrain, with a ditch on each side. In sidelong ground, with only one ditch, it is better to use a **crossfall**.

Figure 19.6 Digging of ditches (copyright International Labour Organisation) (ref. 5).

Stones for penetration macadam may be broken by hand (Figure 19.5) but labour is generally only competitive for sizes of stone above 25 mm.

Working methods

In order to ensure high labour productivity, the use of correct working methods is crucial for all labour-intensive work. Appropriate working methods are described in different handbooks and training materials (refs 2, 5 and 6). Figure 19.6 is an example, illustrating where soil from the ditches should be spread to the correct camber on earth roads.

19.4 TOOLS AND EQUIPMENT

Hand-tools and light equipment are of the same importance in labour-based works as bulldozers, powershovels, graders, and lorries in equipment-based operations. Their quality and design have a big influence on worker productivity and ultimately on both cost and speed of the work.

An appropriate range of tools should be provided so that the most effective implements can be used for different activities and conditions. At the same time, the timely availability of sufficient quantities of tools should be ensured. In order to ensure that tools (and handles) are well designed and ergonomically effective, tool specifications should be explicit and detailed. This also applies to the quality and durability of the items. At the purchasing stage, standard testing procedures should be applied which allow for acceptance or rejection of tools on an objective basis.

The *Guide to Tools and Equipment* (ref. 7), published by the ILO, provides optimum designs and quality standards, and describes appropriate test procedures for a wide range of hand-tools and light equipment. Although these remarks may seem obvious, in most countries these principles are not applied. Tender board procedures are geared to the evaluation of major expensive items rather than to large quantities of hand-tools and light equipment. The introduction of suitable specifications and procurement procedures is crucial. At present, most developing countries do not have appropriate technical specifications for tools and light equipment. The practice of purchasing at lowest prices without giving weight to technical and quality aspects is a main reason for the presence of poorly designed, bad-quality tools in labour-based projects.

Earthmoving

For earthmoving, there is a close correlation between the preferred equipment and the length of the haul.

Headbaskets

For short hauls of 30 m or less, particularly in difficult terrain, the headbasket is a very good choice provided that workers are accustomed to its use. The headbasket is simple and very cheap (Figure 19.7).

Wheelbarrows

For haul lengths greater than about 20–30 m, the use of wheelbarrows increases the productivity of labour compared with head loading. The wheelbarrow is probably the single most useful item of equipment for labour-based road construction. However, many of the wheelbarrows commonly in use are of an inefficient design and are poorly manufactured. The weight distribution is wrong, and the size and type of wheel is ill-suited to construction sites.

Animal carts

Animals can be a most appropriate source of power for haulage. Yet, except in a few areas, they are rarely used for construction. In the countries where animal carts are common, there is a growing appreciation that the scope for improving



Figure 19.7 Headbaskets (ref. 4).

their general design is considerable. The major drawbacks of the traditional vehicles are poor axle bearings, excessive tare weight, the absence of a braking device, poor weight distribution over the cart length, and no system of tipping or bottom discharging. But in many cases the worst part of the design is the crude harnessing device. It has been estimated that a better harness design could often double the output of the animal (ref. 8).

Tractor–trailer

The tractor–trailer combination appears very attractive for labour-based construction. The main reason is that one tractor can be used with several trailers for transporting fill material. Since the trailer can be unhitched, the expensive tractor does not have to stand idle while the trailers are being loaded. However, experience has shown that the trailers need to be very carefully constructed if they are to withstand the rigours of a construction site.

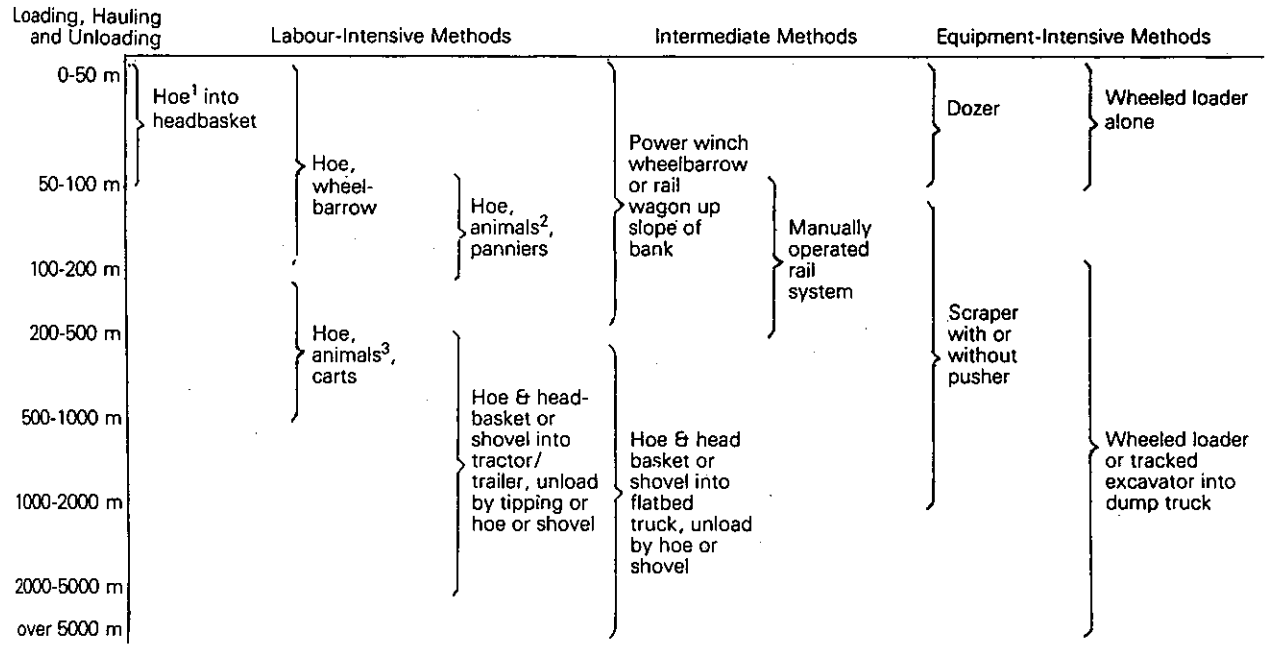
Two-wheeled tractor

The two-wheeled, single-axle tractor has played a vital role in the mechanization of agriculture in Asia. It can be used very effectively for different road construction activities when hitched to different specially designed implements such as trailers, compaction equipment, light scrapers, etc. Its principal advantages are low cost and robust simplicity, which allows their local manufacture and maintenance in many developing countries, particularly in Asia.

Trucks

For long-haul distances it may be appropriate to use trucks. The important point is that the integration of labour-based and equipment-based methods requires special care. It is of little value asking labourers to excavate and load in a similar fashion to machines. The height of loading must be minimal.

The technical and economic feasibility of a range of earthmoving methods is illustrated by Figure 19.8.



Notes: [1]The word 'hoe' is used in the study to describe the back-acting tool used for digging in soft soil and for loading, variously named 'powrah', 'mamti', etc.
[2]i.e. mule, donkey or camel.
[3]i.e. mule, camel or bullock.

Figure 19.8 Available earthmoving methods (ref. 9).

19.5 IMPLEMENTING METHODS

Labour-based road projects may be implemented 'in house' by the road agency, or alternatively using private contractors.

Force account

Force account work is construction undertaken by the government agency using its own resources (direct labour). The true total cost of force account construction is usually above prices bid by well-organized contractors in a competitive situation. In a non-competitive situation, contract costs may exceed force account due to poor management or excessive profit taking. In order to avoid this, it is necessary to maintain a small force account capacity to set standards, and to carry out emergency works and maintenance.

Casual labour

Casual labour is a term used to describe temporarily employed local people. The temporarily employed people do not become a part of the permanent government labour force, and their conditions of work are fundamentally different. This means that, when the project design includes the use of casual labourers, provisions should be made to provide them with a minimum of facilities and security (e.g., first aid provisions on site, clear indications on workers' rights in case of illness or accident, availability of basic facilities such as drinking water, protective clothing, etc.).

Self-help

Self-help is a term used to describe works performed by the local population often with technical assistance from the government, but with no wages. Self-help projects are usually initiated by a local population's interest in infrastructure improvements for which they will be the ultimate beneficiaries. Most self-help projects benefit from some kind of government assistance. The government's contribution to community work can vary from simply supplying tools to the provision of food incentives, partial financing and complete supervision.

Contractors

Contractors are private entrepreneurs who undertake to carry out construction work within a specified time and for an agreed price. Contract amounts can be either negotiated or bid. International lending agencies usually insist that contracts involving their funds be awarded on a competitive basis.

Executing road construction work by contract has a number of advantages. From the executive point of view, contractors are more flexible, not having to abide by strict wage regulations and recruitment policies. They can motivate their staff in a variety of ways which would be impossible for government agencies. They have fewer bureaucratic procedures and consequently fewer delays. Furthermore, they are often able, through an extensive knowledge of the local environment in which they work, to utilize locally available resources in an imaginative and optimum way.

Large contractors tend to be biased towards using equipment-based construction. Small, local contractors are likely to be equipment-poor and therefore more interested in undertaking labour-based construction. In several countries, programmes are being established or are ongoing, which work entirely through local contractors. This type of programme has the multiple objective of introducing cost-effective labour-based methods and developing the private sector.

The transition from direct labour to contract work is difficult to make. In order to transfer the required organizational and management skills to the private sector, training activities must be organized. It is also necessary to develop contract procedures and a payment system appropriate for small local contractors. Working with small contractors also implies that the supervising agency should develop 'in-house' skills to correctly administer, supervise and control the works carried out by contract. In some cases this will involve a third party, the local consultant, who will act on the client's behalf in respect of supervision and control of the works.

Subcontractors

Subcontractors may be used by the main contractor to provide labour or materials only or to undertake specific items of work, such as culverts and bridges.

Maintenance considerations

The future maintenance requirements and methods should be considered when the choice of construction technology is made. If the future road maintenance is intended to be carried out by locally employed people, it is advantageous to also apply construction methods employing labourers living near the road. By doing so, future maintenance workers can be selected from the best construction workers.

19.6 PROJECT ORGANIZATION

The organizational structure of labour-based projects differs from that of equipment-based projects. The size and structure of the organization will vary with the size of the project.

Gang leaders

A proven organization structure is to have one gang leader for every 20–25 workers and one site supervisor in charge of 4–5 gangs. The gang leader would

normally be a worker who has the confidence of his fellows. He would normally leave the project when his gang terminates its employment, although the best gang leaders could be kept for training to become site supervisors.

Supervisors

The site supervisors could be semi-permanent employees kept throughout the project, or they could be permanent employees moving from one project to another.

Senior supervisors

For every 2–4 site supervisors (200–400 workers) a senior supervisor with good experience and technical knowledge of road construction would be needed. A technician with engineering training would be suitable for this post. He would be the person to solve most of the organizational and technical problems on projects consisting of many small and scattered sites.

Engineer

Finally, there should be a road engineer who could direct 2–3 senior supervisors and be assisted by supporting staff for surveying, accounting, supply and stores, work studies and other administrative matters. The following number of supervisory staff would then be needed for a project employing about 1000 workers:

1 engineer
3 senior supervisors
10–12 site supervisors
40–50 gang leaders.

Recruitment

The recruitment of labour can be done in various ways. The involvement of the local administration and community leadership to the maximum extent possible is strongly recommended. Recruitment should be announced formally and well in advance of the start of the works. If the number of workers applying is greater than needed, a ballot system (selection by lottery) should be applied.

Transport

When casual labourers are employed, transport to and from the site is not normally provided. This restricts the area from which labour can be drawn to 5–15 km from the site. As the construction work progresses, the distance to walk will become too great for the workers and new ones will have to be employed. The disadvantage with this system is that productivity will decrease because it takes some time for the labourers to acquire efficient work methods. The alternatives are to provide transport for the workers or to arrange labour camps. Both alternatives are costly and bring with them administrative and organizational problems.

Planning and control

For labour-based works, careful planning is particularly important. Planning should be done using uniform procedures on a monthly, weekly and daily basis. Furthermore, the progress and the use of resources must be controlled. The control process will take place at all levels of the organization hierarchy, and to enable this a flow of reports must be established. A good reporting system is particularly essential in a programme with many small sites as the project management cannot visit each site often. As one moves up the hierarchy of plans and reports, the information will be more and more condensed.

19.7 TRAINING

It is essential to establish training courses specifically adapted to the requirements of labour-based road construction programmes. Such courses should have a large

practical component and emphasize worker management and organizational aspects of the work.

Management

For the higher managerial levels, relevant international courses have been developed for engineers and senior technicians. These courses are organized on an annual basis in Kenya. The ILO also regularly organizes fellowships for managers and decision-makers to successful labour-based programmes in Africa and Asia. Training materials on this subject are also being integrated into the course materials of civil engineering faculties in the universities of developing countries.

Supervisors

The training need is most crucial at the level of senior supervisors and site supervisors. The senior supervisors should be capable of inspecting workmanship, planning and programming, and assuring reporting accuracy. The field supervisors should be fully competent in the work methods to be used, as they will give the daily instructions on site and manage the gang leaders and the workers. Training courses may also be needed for storekeepers, drivers and mechanics. On the maintenance side, special courses will be necessary for maintenance inspectors.

Workers

Gang leaders and workers would not normally receive any formalized training. Their training will be on the job. This sort of informal training is done in the course of the daily duties and should take the form of both instructions and practical demonstrations. However, a number of projects have also started gang-leader training in a more formal and structured manner.

Contractors

If the local private sector is to be involved in labour-based programmes it is necessary to develop appropriate contractor training. Small local contractors normally need to be trained in cost accounting, estimating and bidding, and the comprehension of contract documents dealing with the rights and obligations of the contractor, specifications, drawings, and payment procedures. On the technical side, the contractor's supervisors should receive similar training as the field supervisors in a 'force account' organization. Also, as noted earlier, the staff of the responsible road agency, as well as local consultants, are likely to require training in contract management and supervision.

Training programmes

Development of country-specific training programmes would normally be the responsibility of a project trainer. Courses can be based on general written and audio-visual material developed by ILO. Training must be co-ordinated with the scheduled construction programme, so that the requisite number of trained staff of different levels is available at the right time. If less than the optimum number of trained staff is available it will adversely affect the performance of the ongoing programme. If trained staff do not immediately have the possibility to apply their knowledge and skills, it will lead to demotivation and a waste of training resources.

19.8 PAYMENT SYSTEMS AND MOTIVATION

Proper motivation is essential to successful labour-based construction. Workers paid under a task-rate or piece-rate system (called incentive payment systems) produce a much higher daily output than daily-paid labour.

Daily payment

Daily payment is used: (i) when no productivity data for the major operations in infrastructure construction is available; (ii) for those activities which cannot be

easily measured; and (iii) in countries where incentive payment is forbidden by law or labour agreement.

Task-rate

Task-rate payment consists of a fixed sum paid for a given quantity of work. A daily task is correctly defined if a good worker finishes in 5–6 full work hours. The worker is free to go home as soon as the given quantity of work has been done. The gang leader in charge of the workers ensures that the work has been completed satisfactorily, before the worker is allowed to leave the site. Tasks can be given to individuals or groups of workers.

Payment by task-rate may be unsatisfactory for activities such as gravelling, where labour and machines have to work together, since the labourer's goal is to finish early. If they succeed, the equipment will be under-utilized unless it can be worked independently during the remaining period of the workday. It is also unwise to have labour and heavy equipment on the same site performing identical or interrelated activities, because it demoralizes labourers to observe the machines' much greater output.

Bonus

In Ghana a system has been successfully introduced where labourers who consistently attend work and achieve their tasks over weekly or monthly periods are paid a bonus on top of their task-rate payment.

Piecework

Piecework is a system in which the worker is paid a fixed sum per unit of output, e.g. US$1.00 per cubic metre of hard soil excavated. The worker himself decides how much he will produce and consequently earn. The payment per unit of output has to be determined very carefully and should only be introduced after enough reliable productivity data have been collected. The advantages of this system are that the unit costs are lower than the unit costs achieved with taskwork and that productivity is high. The system works well with migrant labourers who lack the incentive to finish early because they live in a labour camp. Government agencies usually encounter administrative problems in paying on a piecework basis, but contractors are more flexible and able to use this method effectively.

Payment on time

Whatever system is used, it is imperative to pay wages on time. Timely payment to a large, casual labour force is a must in order to retain workers' confidence in the employer, whether it be the government or a contractor. A contractor, however, will be more easily suspected of mismanagement of funds and his work will be seriously affected if payment is not forthcoming at the right time. In contractor development projects, it is important therefore to design payment systems which allow regular worker payment through monthly payments to be deducted from subsequent work payments.

Disbursement procedures

In order to ensure the availability of funds at the right time at the right place, it is crucial to assess existing disbursement procedures. Small labour-based contractors should receive regular payment in order to pay their casual labourers. Particularly during the establishment phase of small road contractors, a system of monthly advance payments deductible from certificate payments should be introduced. Subsequently, a system may be introduced where work certificates can serve as bank guarantees for overdrafts.

Food for work

Some labour-based construction programmes also provide food as part of each payment. This system is particularly used in countries where the general food supplies are insufficient, and where food can serve as a partial payment. In Mozambique, for example, casual workers were able to buy food at subsidized prices up to an amount of 50% of their salaries.

Working conditions

The importance of general working conditions and occupational safety should not be overlooked. The necessity of having drinking water and first aid facilities on site has already been mentioned. Equally, it is essential that the site management does everything possible to establish good relations between workers and the management and among workers themselves. It should always be possible for workers to air their problems and to form workers' associations. The management should always explain decisions. Where possible, group activities such as sports and recreation should be encouraged.

19.9 INSTITUTIONAL ISSUES

Institutional framework

It is important to clearly define the institutional framework in which labour-intensive projects will operate. Often this is not a simple matter because different agencies may be responsible for the planning, construction and maintenance of different classes of roads, cf. Chapter 24.

In general, it is better to modify existing institutions to meet new needs rather than to create new institutions to execute development programmes. In some cases, the responsibility for labour-based road projects has been placed with government agencies other than the highway department, such as local government or rural development agencies, whose principal objectives are to employ the rural poor and to develop local organizations for community construction projects. On other occasions, the creation of a new operational division within the road authority devoted exclusively to labour-based infrastructure construction has proven to be very effective for larger-scale operations.

At national level, the responsible institution will ensure:

- development of project selection criteria;
- preparation of budget requests at central level;
- development of effective payment, planning and reporting procedures;
- preparation for subsequent maintenance activities;
- disbursement of funds to lower levels as appropriate.

Decentralization

The planning, execution and control functions of labour-based projects should be decentralized to the greatest extent possible and local participation in the decision-making should be encouraged.

Labour regulations

Most public road departments, being part of a country's civil service, employ their labour, including unskilled labour, on a permanent or semi-permanent basis. Existing codes and regulations do not fit a situation where casual labour is employed on a large scale. If labour-based road projects are to be implemented 'in house', a new set of regulations concerning wage levels, working conditions and labour standards may have to be created. As far as possible, this should be done before the start of a project and in consultation with the Ministry of Labour to reach a national consensus regarding the terms of employment.

As equipment-intensive methods in many countries have been the prevalent way of constructing roads, staff regulations are naturally geared towards their use with emphasis on academic qualifications. For labour-based work, supervisors need to have man-management and organizational abilities rather than sophisticated technical training. This does not mean that their work carries little responsibility.

The problem is that government regulations often do not recognize that persons with little formal education can take on this kind of responsibility, and therefore their employment and promotion to the appropriate corresponding levels of their more 'permanent' colleagues will be difficult to accomplish. If a government wants to institutionalize labour-based work, this problem must be addressed, for example through the recognition of labour-based training certificates by the Public Service Commission, and the lowering of recruitment standards for labour-based supervisors.

Another essential issue concerns the career development possibilities of trained staff. Labour-based training courses should eventually be integrated into the overall training programme of the road agency and given the same status as other courses. Successfully completed training should become a stepping stone for promotion to a higher rank in the organization.

Contract procedures

Contract procedures for roadworks are generally biased towards the execution of large-sized projects by equipment. Standard specifications, conditions of contract, and methods of tendering all tend to reflect the dominating influence of expatriate consultants and contractors and their equipment-oriented framework. If small firms are to participate in estimating and tendering for small jobs, contract documentation needs to be simple and straightforward. On the other hand, particularly the bills of quantities and the work descriptions need to be sufficiently detailed to guide the small contractors in their estimating. The very size of contracts may also prevent local contractors from bidding. If a contract is broken down to a number of smaller-sized elements the work would be more accessible to local firms.

To be able to tender for road works, contractors are usually required to be registered as road contractors. In order to qualify for this category the contractor must normally possess a minimum of equipment items. For small labour-based contractors it should be made possible to use proven management experience or a training certificate as a pre-qualification requirement. Some countries, e.g., Ghana, have created a separate category for labour-based contractors, allowing them to bid for road works of a certain nature and up to specified amounts.

19.10 PILOT PROJECTS

In countries with no previous experience with labour-based road construction, there may be many obstacles to the introduction of this technology. Most of these obstacles relate to inflexible procedures, preconceived ideas and attitudes. In order to ease inception and implementation of labour-based programmes it is essential to first make the different levels of the government and the administration aware of the socio-economic implications of *not* utilizing local resources.

As a next step, evidence must be shown that labour-based approaches can effectively work in the country concerned. Pilot projects with technical assistance need to be implemented in order to provide a sound base for larger-scale programmes (ref. 10).

Finally, there is the establishment of a large-scale and long-term programme. A transitory period of several years between the pilot phase and the full-scale programme is usually necessary in order to standardize procedures, firmly establish

training programmes and refine and adapt pilot systems to the different requirements of large-scale programmes.

19.11 TWO CASE HISTORIES

19.11.1 Rural access road programme in Kenya

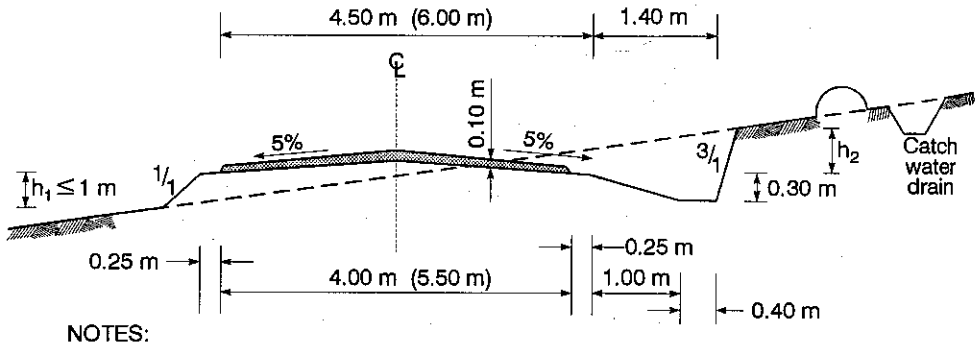
RARP

The Rural Access Road Programme (RARP) in Kenya was the first road construction programme in Africa where labour-intensive construction methods were implemented on a large scale (ref. 2). The RARP was initiated in 1974 when the Kenyan Ministry of Works (MOW), in collaboration with the World Bank and the International Labour Office (ILO), made a study of the feasibility of labour-intensive techniques in Kenya. The British Government agreed to finance the first three rural access road construction units to open up the programme on a pilot scale. An appraisal in 1976 led to an expansion of the programme and, in 1978, 13 road units were operational. In the following years, the number of road units increased to 42. The programme was supported by different donors, including Danida from Denmark.

By the end of 1986, 7600 km of roads were completed. On average, 1600 man-days were used to construct one km of road, but this figure varied considerably with the type of terrain. By constructing good-quality roads at low costs, the RARP proved that labour-based construction methods were economically and technically viable in Kenya.

Standard

The MOW specified that the roads should be of 'all-weather' quality, i.e., that they should be passable during all seasons. Figure 19.9 shows the implemented standard cross-section. Guidelines stipulated that a gradient of 11% should normally not be exceeded and that where a larger gradient was considered necessary, this gradient should not be used for more than 100 m continuously. The



NOTES:

1. Width of earth road formation 4.50 m for less than 30 vehicles per day.

   Width of earth road formation 6.00 m for more than 30 vehicles per day.

2. Height of fill h_1 less than or equal to 1.00 m.
3. h_2 height of cut.

Figure 19.9 Standard cross-sections (copyright International Labour Organisation, ref. 11).

minimum radius of horizontal curvature was 15 m and the minimum desirable radius was 30 m.

Where the height of fill was over one metre, the shoulder was increased from 0.25 to 1.00 m. On very poor soils, the thickness of the compacted gravel course was doubled from 10 cm to 20 cm. In mountainous terrain, where the height of cuts was greater than 2 m, the earthworks were minimized by reducing the width of the road and providing passing bays within sight distance of each other.

Technology

The roads were constructed using mainly local resources. Most construction activities were performed by use of hand-tools. Wheelbarrows and tractor–trailer combinations were used for hauling of material. Compaction was achieved indirectly or by use of hand rammers or light equipment.

Organization

The RARP was a new type of programme for Kenya and for the MOW in particular. To ensure a flexible approach inside the existing structure, a new Special Projects Branch was established within the MOW. This branch was made responsible for the planning, management and organization of the RARP. The management staff consisted of five civil engineers, one mechanical engineer and one construction superintendent.

In 1979, when the administrative systems had been developed and tested and the programme had grown, the programme structure was decentralized. The headquarters staff was cut down to one programme co-ordinator and one construction engineer. In the field, six divisional engineers guided 18 junior engineers who were responsible for the running of 42 construction and gravelling units. The day-to-day running of the individual units was the responsibility of senior supervisors (officers-in-charge).

The average number of casual labourers per unit was 270. The senior supervisor was assisted by a field supervisor (overseer) per 80 casual labourers. Each field supervisor was assisted by a number of gang leaders (headmen), normally one per 15 labourers.

In addition to the supervisory staff, each unit had 4–5 storekeepers, one carpenter, one mechanic and at least one mason.

The personnel were divided into three categories. The supervisors and drivers were permanent MOW employees. Foremen, clerks and artisans were recruited for the duration of the programme but could be dismissed at one month's notice. A problem with the recruitment of those categories was that many candidates who had the necessary abilities and experience could not be employed because they did not have the formal qualifications required by the civil service. The labourers were locally recruited, casual employees. They could be laid off when they were no longer required.

Planning

The systems used in RARP for planning, reporting, expenditure control, and procurement were adapted to the existing structures within the MOW. A lesson learnt was that any programme of an innovative nature should establish management and control systems prior to programme implementation.

Procurement

Procurement of hand-tools was arranged through the existing procurement system of the MOW. During the programme it was demonstrated how crucial it is that sufficient quantities of tools and equipment are available well in advance, and that tools of good design and quality are provided. A great deal of work was done on the development of good technical specifications of hand-tools and their subsequent approval by Government Tender Boards.

Training

The MOW had its own training department. The training department organized a special course programme to cater for the RARP.

Incentive systems

In the early stages of the RARP, the labourers worked under a daily-paid system. After enough data had been assembled, taskwork was gradually introduced. The actual road construction was broken down into two categories of activities, one group which could be carried out by taskwork, the other which could not.

The group of non-taskwork activities included: construction of single-span timber bridges and of drifts, work at camp, setting out and carrying of drinking water for the workers.

Working conditions

In order to establish good relations between workers and the management, weekly site meetings were held by the supervisory staff and the gang leaders. To ensure the occupational safety of the workers, it was common practice to avoid the concentration of large numbers of workers in a small area by measuring out individual tasks in specific areas for each worker. First aid kits were always available in the site camps.

Costs and employment

The average construction costs per km were US$7000 (1985 prices) at a wage rate of approximately US$1.00 per day. At its peak the programme employed some 14 000 casual workers.

Institutionalization

By the mid-1980s, the suitability for labour-based methods for road works in Kenya was accepted by both decision-makers and practitioners. At that time, there was a shift in priority from building of new access roads to improvements and maintenance of the lower category of classified roads (D and E minor roads). The RARP was succeeded by the Minor Roads Programme (MRP) which started in 1986 with the aim of improving selected minor roads and of bringing these under regular maintenance.

By the end of 1993, 3200 km of earth road had been improved. The productivity was 1480 man-days per km and the cost was US$8000 per km.

In 1994 the MRP was being transformed into a labour-based road improvement and maintenance programme for all unpaved roads in Kenya, funded from a Road Fund (Government road taxes, petrol taxes) and – on a decreasing basis – by a donor consortium.

19.11.2 Ghana feeder road rehabilitation project

Ghana has a road network of 14 000 km of trunk roads and 22 000 km of rural roads. The trunk roads are under the responsibility of the Ghana Highways Authority (GHA) while the rural roads are dealt with by the Department of Feeder Roads (DFR). Traditionally GHA and DFR have relied on the private sector for most of the road works.

Aim of project

In 1985 the DFR received assistance in the form of a World Bank credit to improve feeder roads. In addition to its traditional operations, the DFR was interested in developing a labour-based, equipment supported approach because (i) there was a huge backlog of feeder road improvement in many scattered areas; (ii) an under-employed rural labour force was available and motivated to improve the access to the villages. The ILO was requested to define a project which would utilize the available local resources to the maximum extent possible, while relying on small contractors to carry out the works. A three-year pilot project commenced in 1986, the first of its kind in Africa, with the aim of developing small, local

contractors to efficiently apply cost-effective labour-based approaches in rural roads improvement. It also aimed at building up the planning and implementation capacity of DFR.

Contractor training

The project comprised elements of institution building of the DFR, contractor development and maintenance by village labour. The contractor training programme consisted of three main phases: (i) classroom training; (ii) field training; and (iii) trial contracts. Both supervisory and management staff of selected contractors and technical staff from the DFR were provided with intensive technical and management training. During the field training period, the trainees completed the rehabilitation of a 10 km road as a model training site, using labour from local villages. A rotation system was used whereby a trainee was made to supervise a different activity each week, so that he/she had become familiar with all aspects of the rehabilitation work, the management of labour gangs and the overall planning, monitoring and control of labour-based works. During this period, the trainees were also shown how to make estimates and how to prepare bids. Finally, they were familiarized with contractual procedures and documentation.

The first and second phases of the contractor development built up the individual's capacity to apply labour-based methods and to understand the working environment in which contractors have to operate. The aim of the trial contract phase was to develop the capacity of the contractor firms to independently manage a labour-based construction site.

Trial contract

Each of the contractors was awarded a contract of 5 km of road rehabilitation to be completed in a four-month period. This allowed the DFR to assess their performance, and the contractors to manage their own works while having access to technical advice from the DFR and the ILO technical team. At the end of the training period, the contractors were offered a four-year loan through a World Bank credit and a local bank for purchasing their own set of equipment.

Standard contract

Contractors who successfully completed the trial contract in terms of quality and speed were awarded an initial standard contract of 25 km of road. Subsequent World Bank credits and support from Danida and USAID have allowed an expansion of the approach into most regions of the country.

Equipment

Much attention has been paid to the use of tools and equipment appropriate for the site conditions and working methods. Tractor–trailer combinations are used for haulage of materials and heavy-duty pedestrian vibrating rollers for compaction.

Administrative procedures

Streamlining of administrative procedures and prompt payment have contributed significantly to the success of the project. The contractors were advanced 15% of the contract value at the beginning of the contract to cover labour and materials input, and a system of monthly part payments was introduced to allow regular payments to the labour force. The centralized payment systems are expected to be decentralized when sufficient management staff have been posted in the regions.

Incentive systems

The task-rate system, combined with a bonus system to reward regular worker presence, was generally applied for the payment of casual labour. However, a number of contractors used the piecework system for paying workers.

Costs

The cost-effectiveness of this approach has been demonstrated convincingly in terms of quality, speed and cost. In the period from 1986 to 1993, 1190 km of roads were rehabilitated at an average cost of US$14 000 per km at a daily casual wage rate of US$1.70 (1993). The average rehabilitation cost of comparable roads

by equipment was US$16 000 per km. The savings in foreign exchange have been up to 50%. Significant additional benefits were experienced in the rural areas in terms of job creation, local skill development, cash injection and spin-offs to the rural economy, through increased spending on agricultural and industrial inputs.

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PART 8

Maintenance



Grading of gravel roads in Mozambique. (Photo by Bent Thagesen)

20 Maintenance operation

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20.1 INTRODUCTION

Objective

The broad objective of road maintenance is to keep the roads in the original condition as much as possible. However, the resources made available for road maintenance are limited in most countries; industrialized as well as developing countries. This leads to a more comprehensive statement of the objective of maintenance cf. ref. (1):

> 'To conserve, as nearly as possible, the original designed condition of paved and unpaved roadways, and of traffic signs, signals and markings, in a manner most likely to minimize the total cost to society of vehicle operation and accident cost, plus the cost of providing the maintenance itself, under the constraints of severe resource limitations, in respect of skilled manpower, equipment and money, both local and foreign.'

PIARC

In the late 1970s, the foreign aid administrations of France, Germany and the United Kingdom, joined forces to produce a handbook with practical guidelines for road maintenance in Africa. Published in 1982 by the UN Economic Commission for Africa, the book soon became widely known and used far beyond Africa.

In the early 1990s the handbook was reviewed and made more suitable for a wider audience. This work was funded by the Overseas Development Administration (ODA) in the UK and undertaken by the Committee on Technology Transfer and Development under the Permanent International Association of Road Congresses (PIARC). An English version of the new edition of the handbook entitled the *International Road Maintenance Handbook*, divided into four volumes (ref. 2), was published in 1994 by the Transport Research Laboratory (TRL) in UK.

The new PIARC *Road Maintenance Handbook* forms the main basis for this chapter on road maintenance. For detailed information on execution of road maintenance, required personnel, plant, tools, materials and safety equipment the actual Handbook should be consulted.

Highway and Traffic Engineering in Developing Countries Edited by Bent Thagesen. Published in 1995 by E&FN Spon, London. ISBN 0 419 20530 6

20.2 CLASSIFICATION OF MAINTENANCE ACTIVITIES

Maintenance activities may be classified in terms of their operational frequency into:

- routine maintenance;
- periodic maintenance;
- urgent maintenance.

Routine maintenance

Routine maintenance covers activities that must be carried out frequently, i.e. once or more per year. They are typically small scale, or simple, and often widely dispersed. Some of them can be estimated and planned in advance, e.g. vegetation control on shoulders and slopes. Other activities are more difficult to plan in advance, e.g. roadway pothole patching.

HDM

It should be noted that in the HDM model, frequent maintenance activities are termed 'recurrent maintenance'. The term 'routine maintenance' is used only for those types of recurrent maintenance that are independent of the traffic, i.e. maintenance of roadside areas and drainage system (ref. 3 and Chapter 4).

Periodic maintenance

Periodic maintenance describes activities that are needed occasionally, i.e. after a period of some years. They are usually large scale and require more equipment and skilled labour than routine maintenance activities.

Urgent maintenance

Urgent maintenance comprises emergency repair required by flood damage, earth slips, overturned trees, etc.

Maintenance activities and the defects that they treat are discussed below under the headings:

- asphalt pavements;
- unpaved roadways;
- roadside areas;
- drainage systems;
- traffic control devices.

Cement concrete pavements are not covered, as concrete pavements are rarely used in developing countries. In addition, maintenance of bridges is outside the scope of this book.

20.3 ASPHALT PAVEMENTS

Maintenance of asphalt pavements consists of:

- routine activities: sanding
  local sealing
  crack sealing
  filling depressions
  surface patching
  base patching;
- periodic activities: surface dressing
  fog spray and slurry seal
  asphalt overlays
  reconstruction.

Sanding

Bleeding

Sanding is a treatment that may be used when asphaltic road surfacing bleeds. Bleeding is a migration of bitumen to the surface of the pavement. Bleeding is usually caused by too much binder in the surfacing or by an unsuitable binder. Bleeding reduces surface friction and causes the road surface to be slippery.

Sanding

When sanding a bleeding asphalt pavement, a thin layer of sand is spread over the surface. The sand is scattered with a shovel and spread out with a broom. Whenever possible, the sand should be coarse grained with a particle size 0–5 mm.

Local sealing

Local sealing is used to repair more serious bleeding, for sealing local cracks and as a final treatment when carrying out base patching, as described later in this chapter.

Cracking

Cracking may occur in the pavement structure, or in the surfacing only. Longitudinal cracking is often localized along the wheel tracks or along the edges of the pavement. Transverse cracking appears across the whole, or part of the cross-section. Serious cracking will develop into mesh cracking dividing the pavement surface into isolated polygons of different sizes. Mesh cracking is also called block or alligator cracking.

Fine longitudinal cracks in the surfacing may be caused by ageing of the pavement. Deep cracks are usually due to insufficient pavement thickness for the traffic being carried, or poor drainage that allows water to penetrate and weaken the pavement and the subsoil. Isolated transverse cracks are typically caused by shrinkage of a cement treated base. Cracks in asphalt pavements may also be due to poor-quality materials and poor workmanship.

Cracking allows rainwater to seep into the road, and is the beginning of a local or general destruction of the pavement. Only surface cracking and slight cracking in the pavement structure should be repaired with local sealing. Severe cracking in the pavement structure requires base patching.

Local sealing

Local sealing is the application of a surface dressing over a local area. First the area is swept clean; then about 1.5 kg of emulsified bitumen, or 1 kg cutback bitumen, is spread per m^2 surface using a pressure distributor with a spray lance or a watering can. After the binder has been applied, aggregate is distributed by shovel.

When sealing cracks the aggregate should normally be coarse sand. However, when finishing base patching, the aggregate should normally be 6–10 mm chippings. Chippings should be rolled into the bitumen using a small roller or vehicle tyres.

Crack sealing

Instead of using local sealing with surface dressing, closely spaced cracks may be filled with an asphalt slurry. Again the area should be swept clean. A slurry is produced by mixing 20 litres of coarse sand with 6 litres of emulsified bitumen. The slurry is spread out in a thin layer with a wooden board fitted with a handle. An alternative method of sealing isolated cracks is to fill the cracks with hot cutback bitumen.

Filling depressions

This treatment is applied to deal with slight depressions and rutting, slight edge subsidence and small surface irregularities due to shoving. Deep depressions and rutting, deep edge subsidence and large surface irregularities usually require base patching.

Depressions

Depressions are low areas of limited size in the pavement. Depressions usually develop from random local defects in the pavement (low compaction or insufficient pavement strength). Depressions increase the roughness of the pavement and deep depressions may be a safety hazard to the traffic.

Rutting

Rutting, pictured in Figure 20.1, is longitudinal subsidence localized in the wheeltracks of vehicles. The main causes of rutting are inadequate stability of the asphalt material in the surfacing, inadequate compaction of the pavement and insufficient pavement strength. Water may accumulate in ruts during rain and expose the road users to aquaplaning. If water is able to penetrate the surfacing in the ruts, it may lead to cracking and breaking up of the pavement.

Edge subsidence

Edge subsidence, shown in Figure 20.2, usually occurs only where the pavement borders unsurfaced shoulders. The main causes are inadequate, or badly maintained, shoulders and poor drainage that allows water to penetrate into the pavement structure below the edges. Another cause may be a narrow roadway that forces the traffic to drive near the edge. Edge subsidence may lead to destruction of the pavement edges.

Shoving

Shoving, shown in Figure 20.3, is a horizontal displacement of the surfacing. It often occurs on either side of the wheeltracks and also near crossroads where vehicles brake and accelerate. The main causes of shoving are lack of stability and insufficient compaction of the surfacing and/or base. Shoving increases the roughness of the road and may lead to disintegration of the pavement.

Filling depressions

The depressions must be swept clean and dry. A tack coat of hot cutback bitumen at a rate of about 0.5 kg per m^2 is spread using a pressure distributor with a spray lance or a watering can. Cold mix asphalt is placed in the depression and compacted with a hand rammer or a roller. To prevent penetration of water, the PIARC *Road Maintenance Handbook* recommends local sealing of the repair.

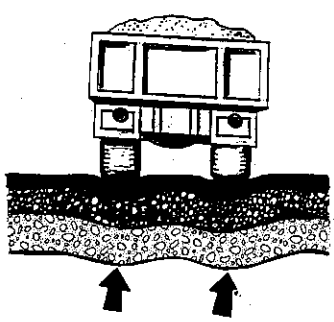


Figure 20.1 Rutting (ref. 2).

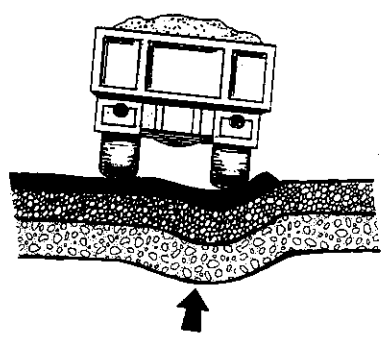


Figure 20.2 Edge subsidence (ref. 2).

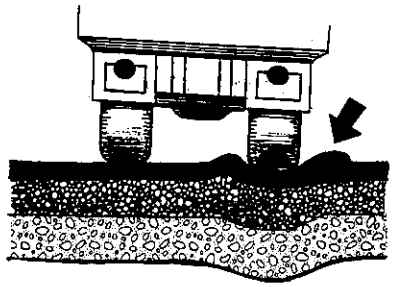


Figure 20.3 Shoving (ref. 2).

Surface patching

Local aggregate loss

Surface patching is used to repair local aggregate loss. Loss of aggregate from the surface of a premixed asphalt pavement is usually due to poor premix quality or poor workmanship. Loss of chippings from a surface dressing may be caused by insufficient binder, use of dirty chippings or insufficient penetration of the chippings into the binder. Loss of aggregate from a surfacing will cause slow disintegration of the layer.

Surface patching

Surface patching may be carried out using the same treatment as for local sealing (surface dressing). An alternative method is to use asphalt premix. The PIARC Handbook recommends use of a cold premix. After spraying of 0.5 kg cutback bitumen per m^2 surface, the cold mix is distributed evenly over the area and compacted with a small roller or a hand rammer.

Base patching

Base patching is used for local restoration of the pavement structure, i.e. to repair severe mesh cracking, deep rutting and depressions, deep edge subsidence and rutting, broken edges, potholes and severe shoving.

Broken edges

Broken edges (Figure 20.4) may result from low shoulders, penetration of water along the edges, insufficient compaction of the pavement edges and narrow roadway. Broken edges are self-perpetuating defects. They may be a safety hazard to traffic.

Potholes

Potholes are local holes in the pavement where pavement materials have been removed by the action of traffic and water. Potholes usually develop in areas

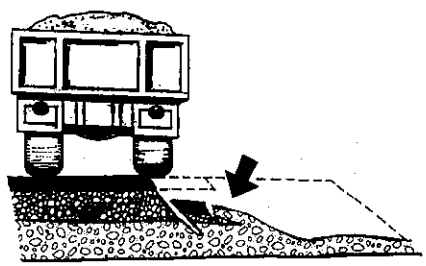


Figure 20.4 Broken edges (ref. 2).

showing cracks, deformations or aggregate loss. In new surface dressings, potholes can develop from random local defects in the surface or base. Potholes drastically increase the roughness of the road and deep potholes are a safety hazard to road users.

Base patching

First, all loose material is removed from the damaged area; the depth of the hole is then increased until firm material is found. The sides of the hole are cut back to vertical and the bottom of the hole is trimmed and compacted flat and parallel with the road surface (Figure 20.5). Any water in the hole must be drained away. The

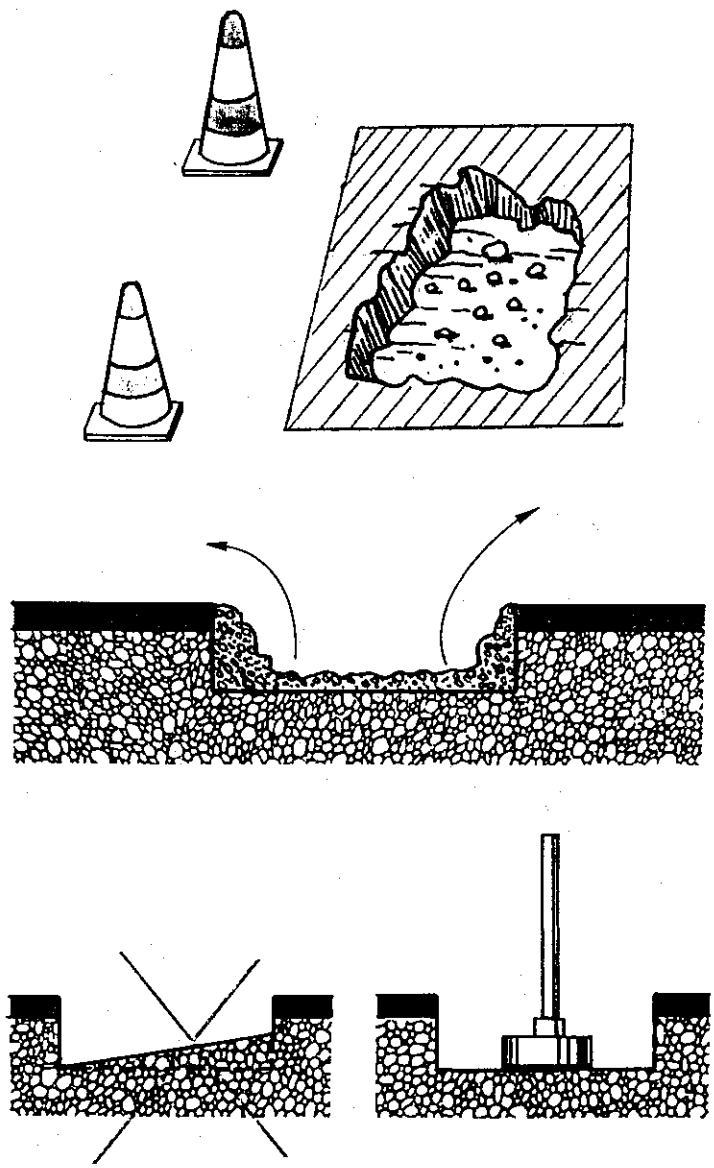


Figure 20.5 Patching of potholes (ref. 2).

prepared hole is filled with material of the same quality as that of the pavement to be repaired, or with an asphalt mix. The material should be placed in the hole in one or more layers and compacted with a hand rammer or a roller, depending on the size of the patch.

Surface dressing

Surface dressing of the complete width of the roadway is appropriate if large areas of the pavement are damaged by bleeding, cracks, slight depressions or aggregate loss. It is also the remedy for streaking and glazing.

Streaking

Streaking is loss of aggregate from a surface dressing in streaks running parallel to the pavement centre-line. Streaking is caused by faulty operation of the spraying equipment.

Glazing

Glazing is due to embedment of chippings in the surface giving a smooth, shiny appearance. The main causes of glazing are wear (not removal of the surface chippings) and embedment of the chippings into the base.

Surface dressing

Surface dressing may be carried out using equipment-based methods as well as by labour-based methods. The quality of labour-based work will be just as good as that using mechanized methods, if the works are properly managed. Information about design and execution of surface dressing is given in Chapter 15.

Before applying a surface dressing over large areas it will normally be necessary to carry out some patching work and repairs to shoulders and the drainage system. If the surface dressing is used to repair cracks and the cracks are due to insufficient pavement or great age of the base, then the pavement needs strengthening with an asphalt overlay or a complete reconstruction may be required, as described below.

Fog spray and slurry seal

Fog spray

Fog spray and slurry seal are options that in certain circumstances may be more appropriate than a surface dressing. A fog spray is a very light film of binder which is sprayed on to the surface of an old, lean asphalt surface in order to hold stone particles in place that otherwise would be picked off by the traffic.

Slurry seal

A slurry seal is the application of a mixture of fine aggregate and emulsified bitumen over the full width of the pavement similar to crack sealing described earlier. The asphalt slurry penetrates and seals surface voids and cracks very effectively. Slurry seals can be prepared in a concrete mixer using a slow-breaking emulsion and then spread on the road by hand. However, the normal technique is to use a mechanized mixer and spreader unit, which enables faster breaking emulsions to be used.

Asphalt overlay

An asphalt overlay is an application of hot, premixed asphalt. Asphalt overlays are used to strengthen old pavements and to strengthen pavements with insufficient thickness. Serious bleeding and severe shoving may also warrant an asphalt overlay.

Strengthening must be preceded by repairs to potholes and edge damage and restoration of shoulders and drainage system. Before applying an asphalt overlay it is necessary to fill in rutting and depressions. Strengthening should always be

properly designed after a thorough examination of the existing pavement, as described in Chapter 16. Information about mix design and construction of hot mixed asphalt pavements is given in Chapter 15.

Reconstruction

Reconstruction encompasses strengthening by multiple-layer overlays and also by recycling of the base and surfacing. Reconstruction is used when this solution is cheaper than an overlay made entirely of asphalt mixture. Reconstruction is also used when complete failure of the construction has occurred. Most road authorities classify reconstruction as a construction activity and not as a periodic maintenance operation.

20.4 UNPAVED ROADWAYS

Unpaved roads are earth roads constructed from the natural soil found on the route, and gravel roads surfaced with a layer of gravel that is stronger than the natural soil.

Maintenance of unpaved roadways includes:

- routine activities: grading
  labour-based minor reshaping
  dragging
  patching;
- periodic activity: labour-based major reshaping
  regravelling.

Grading

Grading is used to remove poor shape, ruts, potholes, corrugations and erosion gullies in the roadway.

Poor shape, ruts and potholes

Poor shape refers to a condition where depressions and lack of crossfall prevent rainwater from draining easily from the surface of the road. Ruts and potholes are types of defects also known from asphalt pavements. All these defects may be caused by inadequate compaction or instability of the materials; and for gravel roads, insufficient thickness of the gravel layer. The defects are self-perpetuating as they will intercept rainwater and allow the water to seep into and weaken the road.

Corrugations

Corrugations are transverse waves on the road surface, probably caused by the bouncing of the wheels of vehicles. Corrugations are generally quite hard and about 25–40 mm in amplitude (ref. 4). The wavelength is usually between 300 and 900 mm and fairly uniform at any particular site. Corrugations only occur on a range of more granular soils. Driving on corrugated roads, if not considered hazardous, is extremely unpleasant and causes mechanical deterioration of vehicles.

Erosion gullies

Erosion gullies are grooves in the road surface caused by rain, and mainly occurring on steep grades. Once erosion gullies have formed, the gullies may quickly grow bigger and destroy the road section concerned.

Grading

Grading is carried out by self-propelled or towed graders. A grader restores the crossfall and removes surface irregularities by returning materials from the sides and shoulders towards the centre of the road. In advance of grading, large potholes and depressions should be patched, areas of standing water should be drained and the side ditches must be cleaned and reshaped. Patching of unpaved roads and cleaning and reshaping of ditches are described later in this chapter.

The grader should work on one side of the road at a time in passes about 200 m long. Where the defects are limited, 2 or 4 passes of the grader may be enough to reshape the road. If the defects are more severe additional passes are needed. In cases where the road is badly damaged it may be necessary to scarify the existing surface to the bottom of any defects and loosen the material for reshaping. Work should be completed on one side of the road at a time. Shoulders are treated as part of the running surface. Whenever possible, rollers should be used to compact the road after grading. Work is best scheduled to follow a period of rain, as the moisture in the material will help compaction whether rollers are used or the compaction is left to the traffic.

Labour-based minor reshaping

Labour-based minor reshaping is an option to grading when maintaining roads with limited defects and low traffic volume. The labourer trims the surfacing material with a pickaxe, hoe or mattock and rakes it to form the specified crossfall. On gravel roads any local depressions and potholes are filled with fresh gravel from stockpiles placed along the road. Whenever possible, the loose material should be compacted with a roller or a hand rammer.

Dragging

Dragging is a simple method used to smooth out minor defects in the road surface. Dragging is usually carried out by towing a specially made drag behind an agricultural tractor. Figure 20.6 shows two different drags, one made from a steel rail and the other made from old truck or tractor tyres.

On low-volume roads, frequent dragging may be used to reduce the need to grade the road. However, dragging will not remove corrugations once they have formed, nor will it restore crossfall and lost material. After a number of dragging operations it is usually necessary to grade the road in order to return materials from the sides and shoulders towards the centre of the road.

Patching

Patching of unpaved roads consists of adding new surfacing material over a relatively small area. Patching may be used to repair deep surface irregularities and to make good local areas with inadequate crossfall. Large-scale patching of gravel roads is called spot regravelling and considered to be a periodic maintenance activity. Patching is not a suitable method for repairing corrugations.

The quality of the material used for patching should be at least as good as the quality of the material already surfacing the road. Top soil containing vegetable matter should never be used for patching. When repairing gravel roads, material

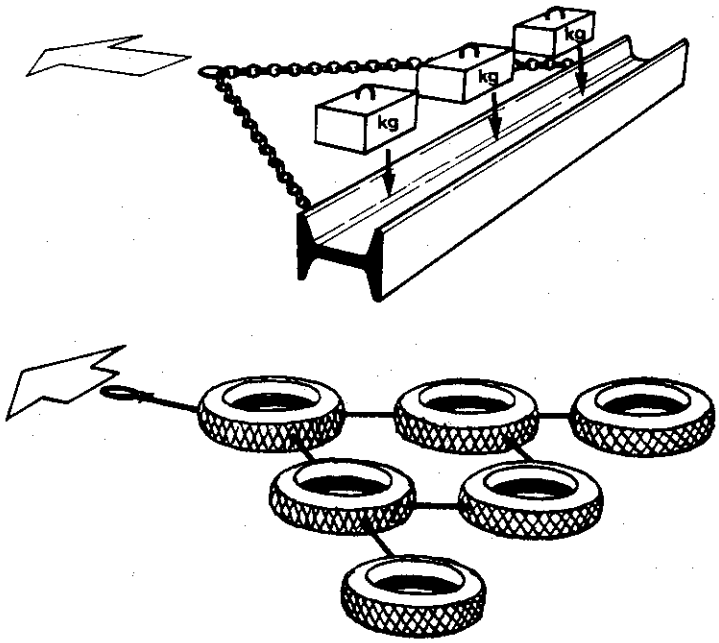


Figure 20.6 Beam drag and tyre drag (ref. 2).

for patching should be dumped at the side of the road near the place where it will be used, without blocking the road or the side drains.

Large potholes are trimmed and the sides cut back to vertical. The hole is then filled with new material in layers of about 10 cm thickness. Each layer is compacted with a hand rammer or a roller depending on the size of the area. Whenever possible, dry material should be sprinkled with water to assist the compaction.

When labour-based maintenance is used, gravel for patching should be stockpiled at every 100–200 m along the road. This is most conveniently done when the periodic maintenance is carried out. A useful place for stockpiling is downslope of turnout drains. Wheelbarrows are used to transport the gravel from the stockpile to the patching site.

Labour-based major reshaping

This method is an option to mechanized grading when complete reshaping of the road cross-section is required and the soil at the side of the road is suitable for constructing the running surface. The method is also used to reshape the road prior to regravelling.

Material is excavated from the ditch and slope area and used to form the camber of the road. Top soil containing vegetable matter should not be used. The fill is compacted with hand rammers or a roller. More material is added until the required cross-section is achieved after compaction. Unsuitable top soil and

surplus material should be removed and dumped well away from the roadside; it must not be discarded along the edge of the road where there is a risk that it will wash down into the ditch during rain.

Regravelling

Regravelling is used on gravel roads to replace lost surfacing materials. Regravelling is also used to correct severe surface irregularities and road sections lacking adequate crossfall.

Material loss

Material loss is caused by erosive wear due to traffic and water. Dry, loose material wears off quicker than cohesive materials and materials containing a little moisture. For gravel roads the material loss inevitably results in a continuously decreasing thickness of the gravel layer. Regravelling is needed before the thickness of the existing gravel layer becomes insufficient. If a gravel road is not regravelled in time, ruts and potholes will quickly appear and the road may soon become impassable.

Regravelling

Before regravelling is carried out, it is important to make any necessary repair or improvements to the crossfall of the road and the drainage system. Also, gravel of a quality that meets existing specifications should be stockpiled at the quarry or borrow pit. The regravelling operation may be executed by equipment-based or labour-based methods.

When using equipment, the road must first be graded and compacted to provide a firm, regular surface on which to work. New gravel is transported by trucks from the quarry or borrow pit to the work site. Dumping of the gravel should start from the far end of the work site, and the material should be placed on one side of the road only and with the spacing necessary to give the intended thickness of gravel over the complete width of the road. The material is now spread across the road using the grader. Water is added with a water tanker. When a correct camber has been achieved, the gravel is compacted with a roller.

When applying labour-based methods, the existing surface is first reshaped to correct crossfall as described above under 'Labour-based major reshaping'. New gravel is usually transported on tractor-drawn trailers. In the borrow pit the gravel should be excavated and stockpiled in a way that allows the trailers to be easily loaded by hand (Figure 20.7). Each tractor should work with two trailers to maximize use of the tractors. On the work site the trailer is unloaded within a rectangle marked with pegs set at the finished road level. The gravel is spread to correct level and crossfall using shovels, hoes and rakes. Whenever possible, the gravel should be compacted with a roller and dry material should be watered prior to compaction.

20.5 ROADSIDE AREAS

Roadside areas consist of shoulders, slopes and other surface areas within the road margin. Paved shoulders and laybys are treated as pavements. Most roadside area maintenance activities can be achieved by labour.

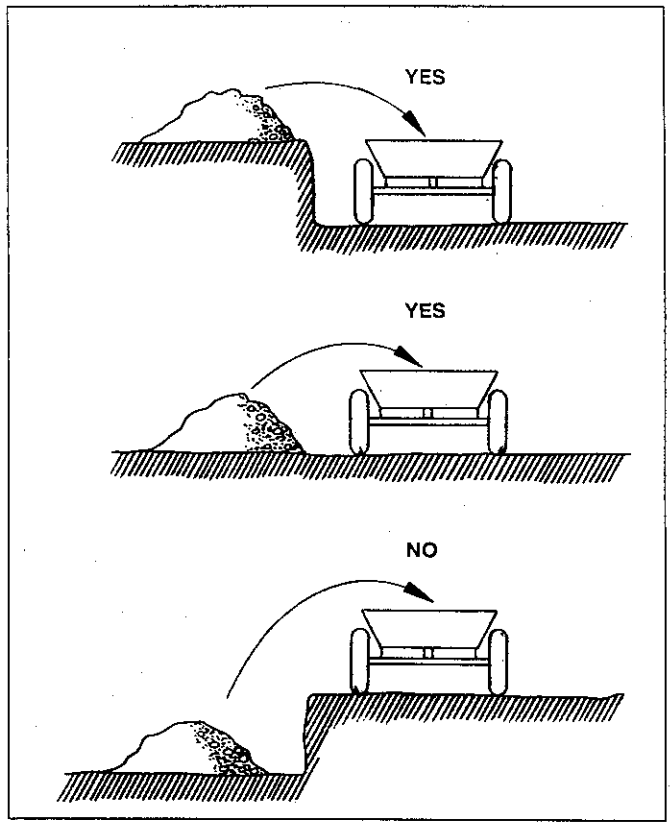


Figure 20.7 Hand-loading of trailers (ref. 2).

20.5.1 Shoulders

The PIARC *Road Maintenance Handbook* lists the following maintenance activities:

- routine activities: removing obstructions
  reshaping shoulders
  vegetation control;
- periodic activities: adding shoulder materials.

Removing obstructions

Shoulders may be obstructed by fallen rock, trees or branches, soil heaps, wind-blown material, abandoned vehicles and debris. The obstructions are a hazard to road users and may prevent flow of water from the roadway to the ditches. They should be removed and disposed of to a safe location.

Reshaping shoulders

High/misshapen shoulders

The surface of shoulders may grow higher than the roadway and the surface may be misshapen if loose material from the roadway is accumulating on the shoulders,

if soil from cutting slopes is slipping on to the shoulders, if vegetation on the shoulder is trapping foreign material, and if traffic on the shoulders causes displacement of material. High shoulders are a hazard to the traffic and may cause water to accumulate along the edges of the roadway and weaken both pavement and shoulders.

Reshaping

High shoulders are repaired by grading or by labour-based reshaping to correct crossfall. When grading shoulders on paved roads, care should be taken not to damage the pavement edges with the grader blade. Shoulders on unpaved roads are graded simultaneously with the roadway. Excess material from the shoulders should be removed and not deposited on the roadway, or in the ditch.

Vegetation control

High vegetation

Grass, weeds and bushes allowed to grow unchecked on the shoulder may cause water and silt to accumulate at the edge of the roadway and may, at worst, impair the sight distance for the road users. In the dry season unchecked vegetation constitutes a fire hazard.

Vegetation control

Vegetation control should be carried out at least once a year. Most of the vegetation may be trimmed by an agricultural tractor equipped with a mower or using a hand-guided power mower. As an alternative to mowing, and in areas inaccessible to power equipment, the vegetation should be trimmed by hand using sickles, scythes, slashers, bushknives and similar hand-tools.

Herbicides

It is not recommended that chemical methods be used to control roadside vegetation. Herbicides may endanger human beings and fauna and can pollute crops and water streams.

Burning

Similarly, roadside vegetation should not be controlled by burning, as the flame and smoke blowing across the road are dangerous for the traffic and the fire can spread and destroy valuable vegetation and crops and animal life.

Adding shoulder material

Addition of new shoulder material is required when the shoulder becomes too low and misshapen by ruts and depressions. This is an activity similar to regravelling of unpaved roadways.

Low/misshapen shoulders

The shoulder may become misshapen and lower than the roadway if traffic runs on it, if the shoulder is eroded by water, if the shoulder has settled, or if the roadway has been repaved leaving the shoulder surface at a lower level. A low shoulder gives inadequate support for the pavement. Water can collect in ruts and depressions and soften the shoulder and pavement edge. The pavement edge can also break away when vehicles run over it. Furthermore, the danger of accidents is increased.

When using equipment, the existing surface of the shoulder is scarified with the tines of a grader. New material is off-loaded on to the shoulder and shaped to correct crossfall and to a level slightly above the final level. On paved roads care must be taken not to damage the edge of the pavement with the grader blade. The shoulder is then compacted with a roller. If the material is dry it should be watered prior to compaction.

When using labour, the existing surface of the shoulder should be loosened with

a pickaxe or mattock. New material is added and shaped using shovels, hoes and rakes. The shoulder is compacted using hand rammers or a manually propelled roller. Whenever possible, dry material should be watered to assist compaction.

20.5.2 Slopes

Slope maintenance includes:

- routine activity: vegetation control;
- periodic activity: erosion control;
- periodic or urgent activity: slip repair.

Vegetation control

Overgrown vegetation due to insufficient grass cutting, bush clearing and tree trimming may reduce visibility for the road users. Tilting trees can fall and block the ditch and roadway. Overgrown vegetation is a fire hazard in the dry season. The activities necessary to control vegetation on slopes are the same as for shoulders, but the required frequency is usually less.

Erosion control

Erosion control on slopes includes different activities: establishing turfing, seeding or riprap on eroded areas, providing berms or channel drains for cuttings, and providing kerb or channel drains for embankments.

Erosion

Erosion of slopes is usually caused by rainwater concentration on the slopes or lack of vegetation cover. If neglected, the erosion may develop into deep ravines and cause a downhill slide of slope material. Eroded material may block the roadside ditch and shoulder.

Erosion repair

Eroded areas may be repaired with grass turfing or seeding. These methods will only be successful if climate and soil conditions are favourable. Small, important areas at bridges and culverts may be protected with riprap (stone pitching). The slope should not be steeper than 1 vertical to 1.5 horizontal. Any rough stone can be used for riprap. The size should be as uniform as possible and the individual stones should weigh 10–20 kg. Heavier stones are preferred if the riprap is not to be grouted.

Slope protection

To prevent future erosion of a cutting slope, a soil berm or a cut-off drain can be built along the top of the slope to prevent surface water from flowing down and eroding the face of the cutting. Berms and cut-off drains should be constructed to lead all water to a safe discharge location.

For embankments, a kerb or channel drain may be built at the edge of the roadway or the back of the shoulder. The kerb may be of dressed stone or concrete. If the gradient falls throughout the embankment the kerb or channel drain may discharge at the transition between fill and cut. If there is a low point on the embankment, a chute or a cascade will be required to safely discharge the water down the slope. A chute is an inclined pipe-drain constructed in or on a slope. A cascade is a channel with a series of steps, to take water down a steep slope, and dissipate some of the energy of flow.

Slip repair

Slip repair comprises those activities required to clear slips and to stabilize slopes in order to prevent future slips in the side slopes of embankments and cuttings.

Slips

Slips or slope instability are usually caused by adverse ground conditions, or water, or both. The slope may be too steep for its height and soil type. Water may be penetrating the slope from above or groundwater pressure may be present.

Slip repair

All slipped soil should be excavated from the roadway, shoulder and drain and removed to a suitable dump site. However, slip repair is a dangerous activity and it is often advisable to reduce the slope angle before clearing the existing slip material. Where a cutting face has slipped, an option is to reduce the angle of at least the upper part of the slope.

A gabion retaining wall may be used to stabilize the base of the slope. A gabion is a stone-filled steel mesh cage. A gabion retaining wall is free draining and sufficiently flexible to allow for further small slip movements. Where the foundation is stable, an option is to construct a masonry or concrete retaining wall.

20.6 DRAINAGE SYSTEMS

The drainage system consists of:

- side drains (ditches), turnouts (mitre drains) and cut-off drains;
- culverts;
- fords, drifts and causeways;
- drainage pipes and manholes.

Maintenance of the drainage system includes the activities listed in Table 20.1. Most drainage maintenance can be achieved by labour-based methods.

Table 20.1 Activities included in the maintenance of the drainage system (ref. 2).

Elements	*Routine activities*	*Periodic activities*
Drains	Clearing and cleaning Reshaping and deepening Minor erosion control	Providing new turnouts Major erosion control
Culverts	Clearing and cleaning Erosion repair Cracking repair Walls and apron repair	Repair of invert Construction of outfall basin Reconstruction
Fords, drifts and causeways	Clearing Repair Replacing of guide posts	
Drainage pipes and manholes	Clearing of pipe and manhole Replacing of manhole and grating Clearing of manhole area Cleaning of catchpit sump	Relay drainage pipe

20.6.1 Drains

Clearing and cleaning

The object of clearing and cleaning drains is to remove all obstructions which could possibly interfere with water flow. Obstructions can include loose silt, sand, gravel, boulders, weeds, bushes and fallen trees. Obstructions should be removed and disposed of well away from the roadside, so they will not fall or wash back into drains. Where a ditch has the correct depth and profile, with grass cover and no erosion, it is advisable to cut the grass short only. This will leave the roots in place to bind the surface together.

Reshaping and deepening

The object of reshaping and/or deepening a drain is to re-establish the correct cross-section and to control silting and ponding.

Destroyed cross-section, silting and ponding

The cross-section may be destroyed due to vehicular or animal traffic or due to collapse of the ditch sides. Silting and ponding (standing water) occur where the invert slope of the drain is so flat that the water cannot flow at sufficient speed. Ponding may also be caused by a drain cross-section which is too small.

Reshaping and deepening

A grader can be used to reshape and deepen V-shaped ditches. Trapezoidal side drains are best maintained by labour using the hoe, shovel and ditch template. In flat areas the gradient of the drain should be carefully checked. Material excavated must be removed and spread well clear of the drain so that later it cannot fall or wash back into the drain. Material from the ditches should normally not be spread on the running surface of the road as it is unsuitable as a surfacing material due to its high content of fine-grained material.

Minor erosion control

Erosion

Erosion of drains occurs where velocity of flow of the water is too great. Erosion is often seen at drain sections laid at steep gradient, on sharp bends without erosion protection and at drain outfalls.

Erosion repair

Unlined drains that have been damaged by scour of the invert and sides may be repaired by filling with soil and turfing, provided that the climatic conditions are favourable. To avoid future erosion, simple scour checks may be constructed of wood pegs or stones. All scour checks should have an apron downstream built of stones or grass turves pinned to the ditch invert with wooden pegs. Lined drains, where the linings have been damaged, should be repaired as soon as possible.

To avoid future erosion, drain sections with a steep gradient may be realigned to follow contour lines more closely. Sections with sharp bends may be relayed to smooth curves or specially precast curved sections may be installed. Drain outfalls may be extended to reduce the speed of the water when leaving the ditch. The gradient should ideally be between 2% and 5%. Areas downstream from outfalls may be protected by turfing or stone pitching.

Provide extra turnouts

Extra turnouts may be required where water ponds in the side drains, where drains

are hydraulically overloaded, and where erosion occurs. Turnouts are desirable at spacings as close as 20 m on some gradients. If water cannot be discharged from a drain over a distance of 200 m, it may be necessary to line the drain or construct an extra culvert.

Major erosion control

Major erosion control includes lining of ditches, where they are frequently damaged, and construction of chutes or cascades in places where large volumes of water have to be taken down slopes. The lining may be constructed with stone masonry, precast concrete tiles or precast drain units. Cascades are constructed *in situ* from stone masonry or concrete.

20.6.2 Culverts

Cleaning and clearing

Blockage

Culverts may be blocked by sand or floating vegetation and debris. This happens particularly when the invert slope of the culvert is too flat and when the invert level is lower than the bottom of the downstream ditch. These are often construction or design faults. The result of a blockage is that the intended waterway opening is reduced to the extent that floodwater cannot flow. The floodwater will back-up or pond on the upstream side of the culvert and may eventually overflow the road embankment. The road is then in danger of being severely eroded and washed away.

Clearing and cleaning

Material and debris blocking culverts must be removed and spread or dumped where they cannot impede water flow. In addition, the upstream approaches and the downstream area must be freed of any obstructions. If floating debris is a recurrent problem, the provision of a debris rack should be considered.

Cleaning of culverts with openings smaller than 1 m is a particular problem. Sometimes small culverts can be cleaned by pulling a cable or rope through, to which is attached a bucket.

Erosion repair

Erosion

A commonly occurring defect is erosion of the stream bed at the culvert outlet. This happens when the culvert invert has been constructed with excessive gradient so that the velocity of water flow is too great, or when the culvert invert has been constructed with too flat a slope and an excessive drop at the outfall. These defects are normally design or construction faults, and if neglected the stream bed is washed away and the downstream headwalls and wingwalls, and even sections of the culvert and road embankment, can collapse.

Repair

Where only light erosion has taken place, the eroded area should be filled with stone blocks of about 30 cm size to produce a rough energy dissipater. The block pitching should preferably extend beyond the eroded area. Where stone is not available, logs or jute sacks filled with soil and cement can be used.

Cracking repair

Cracking in headwalls, wingwalls and the main structure is usually due to settlement of the foundation soil below the culvert. If the settlement is small and limited in extent, only minor cracking will result and this will hardly affect the functioning of the culvert. However, cracks should be repaired as soon as possible to reduce water penetration of the foundation and further settlement. To repair cracks, first they are cleaned carefully with brush and water and all loose material is removed. Then the cracks are wetted and filled with cement mortar.

Walls and apron repair

Where parts or all of the headwall, wingwalls or apron of a masonry or brick headwall have been damaged by erosion or settlement, it is necessary to repair the damage as quickly and effectively as possible.

The settled or damaged section of the walls and apron should be removed and the underlying soil compacted. Then the walls and/or apron should be rebuilt using materials similar to the original. When the repaired walls are strong enough, the backfill should be restored.

Repair of invert

Corrosion

The invert of steel culverts, especially old steel culverts, is often rust corroded because the protective galvanizing has worn away. Repair should be carried out as soon as the surface starts rusting. If neglected the damage may accelerate and eventually develop into collapse of the structure.

Repair

Invert repair is dry-season work. Steel culverts with superficial rust damage are cleaned with a steel brush. After cleaning, a thick coat of bitumen is applied over the lower half of the culvert.

If the invert is seriously damaged, a plain cement concrete slab should be poured over the complete length of the invert. The inlet and outlet aprons should also be concreted and adjusted to the new levels of the culvert invert.

Construction of outfall basin

If the outfall of a culvert suffers from continual erosion, an outfall basin (catchpit) should be constructed (Figure 20.8). The outfall basin will reduce the energy of the water and decrease the risk of downstream erosion. An outfall basin may be constructed of stone masonry or concrete.

Reconstruction

Culverts that have collapsed or are beyond repair should be reconstructed. Where erosion problems exist in the side drains due to excessive water flow, new culverts may be needed to discharge excess flow to a less overloaded receptor. Consideration should also be given to replacement of culverts having inadequate diameters (less than 60 cm). Furthermore, reconstruction may be required if the culvert and outfall repeatedly silts up because the culvert has been constructed too low. The

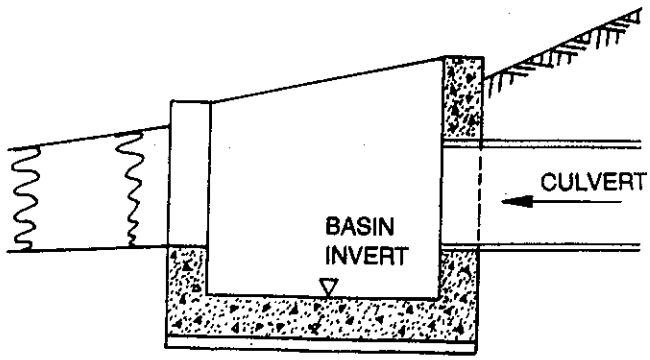


Figure 20.8 Outfall basin (ref. 2).

level may then need to be raised for a distance to accommodate raising of the culvert.

20.6.3 Fords, drifts and causeways

Clearing

Water will deposit soil and debris on fords, drifts (paved fords) and causeways (vented drifts) from time to time. This creates a hazard to the traffic and exposes the crossing to erosion. Material and debris covering fords, drifts and causeways should be removed and deposited downstream, well clear of the crossing.

Repair

Drifts and causeways are often damaged by cracks, settlements and erosion due to water turbulence. These defects should be repaired as quickly as possible. If not, the defects may spread and cause undermining and disintegration of the drift slab or the causeway structure.

Cracks should be cleaned and filled with a bituminous mortar. Erosion cavities upstream or downstream of the drift or causeway should be filled with large stones or riprap. Serious, or recurring erosion may be controlled with gabion mattresses.

Replacing of guideposts

Guideposts marking the water crossing may be damaged or missing due to accidents, flood damage or vandalism. This is dangerous for the traffic that may accidentally drive into deep water when the crossing becomes submerged during floods. Missing or damaged guideposts should be replaced before heavy rains.

20.6.4 Drainage pipes and manholes

Drainage pipes and manholes are mainly used in built-up areas and are often part of the sewerage system. For maintenance of drainage pipes and manholes reference is made to the PIARC *Road Maintenance Handbook*.

20.7 TRAFFIC CONTROL DEVICES

Traffic control devices include road signs, guideposts, kilometre markers, guardrails and pavement markings.

In order to serve their intended function, traffic control devices should be kept in a condition similar to that at the original installation. Maintenance of traffic control devices include:

- routine activities: cleaning
  repainting
  repairing in the workshop;
- periodic activities: replacing guardrails
  pavement marking
  replacing traffic signs
  repairing or relocating kilometre markers.

For further information, see the PIARC *Road Maintenance Handbook.*

20.8 PRECAUTIONS

Before starting any maintenance work, precautions should be taken to ensure the safety of both road workers and road users.

Safety vests

Yellow or orange coloured safety vests should be worn by all workers. Vehicles and equipment should also be painted yellow or orange and carry red and white striped markers front and rear. The vehicles should work with headlights activated and, where possible, carry yellow flashing lights or flags.

Advanced warning

Ahead of the work site in both traffic directions, signs giving advanced warning of danger should be located. Along the length of the roadworks, traffic cones should be placed to protect the site from traffic. At the end of the roadworks for both directions a sign should be placed indicating the end of restrictions.

Traffic control

If the traffic can only pass the work site in one lane, a barrier should be placed at each end, and traffic controllers should stand next to the barriers and operate a reversible stop/go sign. On long work sections and sections with restricted overview, hand radio sets or intermediate traffic controllers are required to transfer the stop/go instructions to the traffic controllers. On large work sites portable traffic lights may be more expedient. In some cases it may be necessary to close the road temporarily and divert the traffic.

On roads with low traffic volumes it may be appropriate to use a simpler system of traffic control. The type, number and exact location of the different traffic signs and traffic barriers depend on the road class and the type, extent and duration of the roadworks.

In order to uphold respect for traffic signs, it is important that all temporary warning signs are removed immediately after the roadworks have been completed.

20.9 IMPLEMENTATION

Road maintenance may be undertaken by force account, contractors or self-help groups (ref. 5).

Force account

Traditionally, most maintenance activities are undertaken by the road agency itself using direct labour, also called force account. Force account may be used for implementing both equipment-based and labour-based maintenance. The work may be organized as an area system or a patrol gang system.

Area system

The area system concentrates all the maintenance resources at one location. Areas usually include 100–300 km of road and employ a maintenance force of approximately 10 men. The area is inspected regularly and crews are made up daily to handle the various maintenance tasks as determined by the inspections. This system is most suitable for a close network of roads within a relatively limited area. It is easy to administer because the crew is based at one site only.

Patrol gang

The patrol gang system includes several units, each consisting of three or more men and a motor truck. Each patrol inspects and maintains up to 150 km of road. A field supervisor directs a number of units and allocates tasks to combined mobile gangs, once one patrol unit has localized difficulties.

Contractors

Reliance on private contractors for road maintenance is increasing. Evidence shows that contractors can often provide labour and equipment at a lower cost and with less delay than government organizations. It is obvious that periodic maintenance activities, being rather similar to construction activities, are well suited for contracting. However, experience has also shown that in many developing countries, routine maintenance by labour-based methods can be successfully handled by single contractors, also known as lengthmen.

Lengthman system

The lengthman system of maintenance involves a contract between a local individual (the lengthman) and a government agency. The lengthman receives payment to carry out all routine maintenance over a fixed length of road using hand-tools furnished by the government. The contract stipulates that payment can only be made if the road section is in a good state of maintenance and if it is clear that regular maintenance has been carried out. This system operates satisfactorily in populated areas where the lengthman lives close to his assigned road section.

Lengthman maintenance is suitable for all types of rural roads. The lengthman is motivated to do a good job because he can earn local status and recognition as a valued community member. A large group of lengthmen is assigned to one supervisor who is usually also in charge of a supplementary mobile gang that can handle larger tasks. One supervisor with a motorcycle may handle inspection and wage payments for 200–300 km of low-volume roads if all the roads lie within a radius of about 100 km from his base station.

Self-help

Self-help maintenance is maintenance undertaken voluntarily, without payment, by the local population. Self-help maintenance is only relevant for rural access roads where the local populations will suffer directly from inadequate road maintenance. Self-help maintenance is most likely to succeed when the road was constructed by self-help. Few self-help maintenance operations exist without some government assistance.

Institutional issues

Implementing methods for road maintenance is closely linked to the institutional arrangement of the road agency. Institutional aspects of road maintenance are discussed in Chapter 24.

Case history

The maintenance system that has been adopted for the gravel roads constructed under the Rural Access Road Programme (RARP) in Kenya (Chapter 19) relies on the lengthman system. The workers are contracted immediately after the road, or section of the road, has been completed. In general, the workers are former employees of the RARP construction unit. Consequently, they have a knowledge

of road construction and of the standard to which the road has to be maintained. Moreover, since the workers are known to the RARP supervisory staff, it is possible to offer the contracts to those workers who have shown responsibility and diligence during construction. It is ensured that the workers live in the immediate vicinity of the section of the road they are to maintain.

Lengthmen

An average of 1.5 km of road are allocated to each lengthman based on the assumption that routine maintenance of a road section of this length requires approximately 12 man-days of work per month. This enables the workers to spend the remainder of their time on their farms.

Tools

Each lengthman is issued with a selection of the following tools:

- hoe
- shovel
- culvert-cleaning shovel
- rake
- grass slasher
- hand rammer
- file
- bushknife
- wheelbarrow.

Every 200 m, a stockpile of gravel is provided for patching of potholes. Periodic maintenance, such as major erosion control and reconstruction of culverts, is carried out by a special task force. When regravelling, tractors and trailers are provided for hauling of gravel.

Payment

The lengthmen are paid a fixed monthly salary of 12 working days at the current rate for casual labour. The payment is made provided that the road has been inspected and found satisfactory. A supervisor supplied with a motorcycle manages inspection and payment of 100 lengthmen. If a road section is found in an unsatisfactory condition due to neglect by a lengthman, the payment is withheld. If the road section is in a proper condition the following month, the lengthman is paid for two months. Otherwise he is dismissed.

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Maintenance management

21

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21.1 THE MAINTENANCE PROBLEM

21.1.1 The present situation

Inadequate road maintenance is a serious problem in most developing countries. Taking the Sub-Saharan African countries as an example: a recent survey estimated that, in this region, some 130 000 km of paved roads, 350 000 km of gravel roads, and 425 000 km of earth roads and tracks had been constructed up to 1985 (ref. 1). The survey indicated that the major part of this network was in bad condition. No significant routine maintenance was carried out on earth roads, about one-fifth of the gravel roads were not maintained, and even paved roads were poorly maintained with disastrous consequences for the economies of the African countries concerned. The conditions are not much better in many developing countries in other parts of the world.

21.1.2 Causes of the problem

Inadequate road maintenance may be due to several reasons. Insufficient funds, including a lack of foreign exchange, shortage of qualified staff, absence of machines and spare parts, deficient institutional arrangements, and poor management capability are normally held responsible for the problems.

Funds

The most common reason given for inadequate maintenance is the difficulty in securing the necessary funds. It is true that only limited funds are available for road maintenance in most countries, but it is also apparent that the funds allocated for maintenance are generally not utilized very efficiently. Often funds are diverted to non-maintenance projects, they are used on activities with political rather than economic priority, or they are used to support large labour forces which are unproductive (ref. 2).

Foreign exchange

Lack of foreign exchange is also a problem in many developing countries. In particular, lack of foreign exchange impedes the purchase and operation of mechanical equipment. The adoption of labour-based technology is seen by a

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number of developing countries as a viable solution to this problem. Labour is often readily available and labour-based techniques are appropriate for a number of maintenance activities. Labour-based methods are likely to be cheaper than those based on equipment where the labour rate is less than about $4–6 per day.

Staff

The shortage of qualified and skilled staff is a fundamental problem in most developing countries. The recognition of this situation has led to new development projects often incorporating training of local staff as an important component. Training should be an on-going feature of employment in the road organization, so that competent staff are available to take over when experienced personnel leave. This is discussed in more detail in Chapter 25.

Equipment

Absence of equipment is linked to lack of foreign exchange and shortage of qualified staff to maintain the equipment. However, the allocation of foreign exchange and the presence of qualified mechanics does not necessarily secure an efficient utilization of existing equipment. In several studies it has been observed that the utilization of existing tractors and motor graders is down to 30% and 10% respectively because of bad management and bureaucratic procedures in the procurement of spare parts. In an efficient road organization, tractors and graders should be in use more than 50% of the work time.

Institutional and management issues

Studies of road operation in developing countries have consistently highlighted deficient institutional arrangements and management practices as major reasons for inadequate road maintenance. Most road agencies have too many responsibilities. Too much emphasis is often put on execution, whilst planning, control and evaluation are neglected. These aspects of road maintenance are dealt with in Chapter 24.

21.1.3 Maintenance capability and management

Maintenance capability

The maintenance capability of the responsible road agency should be a key criterion when appraising new road projects. If it is likely that the future road will not receive adequate maintenance, then it is unlikely that future benefits will be achieved, and the proposed project should not be implemented. In such cases, it would be more advisable to improve the maintenance of existing roads. Standard checklists are available (ref. 3) for the evaluation of the maintenance capability of a road agency. An indication of capability may also be obtained from a survey of the condition of existing roads.

Management

Inefficient use of limited funds for road maintenance is often closely linked to poor maintenance planning. It has, therefore, been concluded by many agencies that the improvement of management practices is a prerequisite to improving maintenance capability. Such an approach usually needs to be supported by the implementation of maintenance management systems. The approach to the management of road maintenance and the use of management systems are discussed in the following sections of this chapter.

21.2 APPROACH TO MAINTENANCE MANAGEMENT

Because of the intense pressure to make optimum use of limited resources, maintenance management presents a greater challenge to the road engineer than

maintenance techniques. Most maintenance techniques are relatively straightforward and easily acquired through well-organized training. Maintenance engineers must have a detailed knowledge of all maintenance techniques, but supervision of the majority of maintenance activities should be delegated to technicians or foremen. The maintenance engineer should utilize his time better by planning, programming, budgeting and monitoring these activities.

Objectives

Maintenance management has the following objectives:

- to encourage the use of a systematic approach to decision-making within a consistent and defined framework;
- to provide a common basis for assessing maintenance needs and resource requirements;
- to encourage the adoption of consistent maintenance standards;
- to assist in the effective allocation of resources;
- to encourage regular review of policies, standards and the effectiveness of programmes.

In simple terms, maintenance management aims to get the *right* people, materials and equipment, to the *right* place on the road network, to carry out the *right* remedial or preventative work, at the *right* time.

Management cycle

Maintenance management is normally undertaken as a cycle of activities, carried out on an annual basis, in the following steps:

- setting of maintenance policy, objectives and standards;
- classification and preparation of road register;
- assessment of maintenance needs;
- calculation of resource requirements;
- assessment of priorities when resources are constrained;
- scheduling and executing of works;
- monitoring of performance.

The remainder of this section deals with each of these in turn.

21.2.1 Maintenance policy, objectives and standards

Policies are the key issue in the management of highway maintenance. They define the broad level of service which the highway authority intends to provide in terms of level of comfort, safety, economic benefit and the cost of provision. One example of a maintenance policy is given in the introduction to Chapter 20. As an another example the following road maintenance policy has been adopted by the Ministry of Surface Transport in India:

'To maintain and operate the highway system in a manner such that:

- comfort, convenience and safety are afforded to the public;
- the investment in roads, bridges and appurtenances is preserved;
- the aesthetics and compatibility of the highway system within the environment is preserved;
- the necessary expenditure of resources is accomplished with continuing emphasis on economy.'

Whereas policy statements are fairly broad in their scope, objectives quantify these to be more explicit. For example, an objective reflecting the 'comfort,

convenience and safety' aspect of the above policy might be that 'pot-holes will be repaired within one week of being reported'.

Standards then provide the thresholds that trigger action. For example: the standard would define what is a pothole, i.e. 'localized very severe ravelling extending to greater than the full depth of the wearing course'.

Policies, objectives and standards should be agreed initially, and then should be monitored on a regular basis to ensure their continued applicability, making changes when this proves necessary.

21.2.2 Classification and preparation of road register

Classification

Levels of service and maintenance standards will vary depending upon the nature of the road and traffic. Roads are, therefore, normally allocated to categories which form a hierarchy. A common set of hierarchies commonly used in developing countries is:

- arterial
- collector
- access.

Roads register

Information relating to classification and condition of the roads must be related to the geographic location of any item on the highway network. Thus, a system of network referencing is fundamental to highway management. For management purposes, the network is usually broken down into a series of links or sections, each defined by a unique label. The list of links or sections provides a roads register which defines the entire highway network. The start and finish of links and sections must also be identified physically on the road, and marker posts, often at kilometre intervals, can be conveniently used for this.

The roads register will also contain inventory information which lists, against each link or section, details of the physical characteristics of the road. All of this information is used as reference for the planning system. The register should typically contain the following information:

- Alignment: the chainage of characteristic points in the alignment, including location of crossroads, culverts, bridges, and sharp curves; radii of sharp curves may also be recorded;
- Longitudinal profile: the chainage of vertical curves and, optionally, the gradients on hilly stretches;
- Cross-section: the width of the carriageway and shoulders; information as to whether the road is provided with side ditches and hard shoulders;
- Pavement: the type, thickness and, if possible, the age of the pavement on the carriageway and on the shoulders;
- Structures: the type and dimensions of major culverts and bridges;
- Furniture: information on road signs and guardrails;
- Land use: information about the soil type along the road (clay, sand, rock, etc.) and the land use (town, village, woods, farmland), and location of identified deposits of road materials.

Register

The roads register should be as simple as possible and not overloaded with unnecessary information. Data for a simple road register may be collected by driving slowly through the road network and stopping for measurement of characteristic cross-sections. Chainages can be recorded on the car's trip meter. Degrees of horizontal curvature can be determined with a compass, or in relation to the turning angle of the car's steering wheel. Gradients can be measured by means of a simple fall meter. The collected information can be recorded in tables or on schematic road plans, but most planning systems for road maintenance are based on the use of a computerized roads register.

Updating

The preparation of a roads register is a once-only activity. However, it is very important that the register is kept up-to-date. If information on changes to the network, new reseals, overlays and reconstruction works is not fed into the register, its usefulness is reduced and maintenance planning is made more difficult.

21.2.3 Assessment of maintenance needs

Condition survey

Defects

In order to assess the maintenance needs of a road network it is necessary to register the present defects. A condition survey is usually based on a visual inspection. For major roads it is good practice to supplement the visual inspection with mechanized measurements. Mechanized data are more repeatable, reproducible and, generally, can be collected more efficiently. Mechanized data collection provides scientifically based techniques to assist in the determination of causal factors of defects, to monitor changes in condition, to assess strength and to help determine appropriate treatments. Visual inspections are normally used for registration of all conspicuous defects of pavement, shoulders, ditches, culverts, slopes, and road furniture, cf. Chapter 20. Mechanized measurements are usually limited to the pavement.

Visual inspection

A visual condition survey of the road network should be carried out at least once a year to assist in determining maintenance needs for the next budget period. In countries with a tropical climate, the drainage system should be inspected twice a year: once in the rainy season and once in the dry season. The evaluation of the condition in the rainy season is especially important as the drainage system can only be evaluated satisfactorily when there is water present.

It is useful if the engineer responsible for the road maintenance participates personally in visual inspections. This will ensure that he is able to plan maintenance works effectively, based on his familiarity with the road network, and is able to control the quality of work executed.

Recording

Normally when inspecting a road section, the road is divided into subsections, typically 100 or 200 metres in length. The road register marker posts are used as a reference. For each distress mode, the extent and the severity of the defect are recorded. Recording of defects should be supplemented by an assessment of their possible causes. Knowledge of causal relations is fundamental for the selection of appropriate repair methods. Figures 21.1 and 21.2 show examples of forms used for inspection of paved and unpaved roads respectively.

Mechanized measurements

Mechanized measurements include recording of functional as well as structural parameters. The most widely used functional parameter is the roughness.

PAVED ROAD INSPECTION REPORT			DATE	23 APR. 1986						INSPECTOR	AHMED					
ROAD NO.	A23		SUB-SECTION NO.	5				START km	187 + 200			PAVEMENT TYPE	SD + GRAN			
Distance			187+	200				+300								ACTION REQUIRED
			0	25	50	75	0	25	50	75	0	25	50	75	0	
LEFT	Side drain /turnout	Silt														
		Scour		/	/	/	/	/	/	/	/					Filling
	Shoulder	Deform														
		Scour		/	/	/	/	/	/	/	/					Filling
		Vegetation														
	Edge step						/									Edge patching
	Edge damage					/	/	/		/						
CARRIAGEWAY	Rutting					5	5	10	10	15	15					
	Cracking	Wheeltrack	2	2	5	8	10	5	10	25					67	Further
		Other				5	2	10	5	10					32	investigation
	Stripping/fretting															
	Pot-holes															
	Fatting-up/bleeding															
	Lane marking															
RIGHT	Edge damage															
	Edge step															
	Shoulder	Vegetation				/	/	/	/							Grass cutting
		Scour														
		Deform.														
	Side drain /turnout	Scour														
		Silt														
Notes/comments																

Figure 21.1 Inspection form for paved roads (TRRL Overseas Road Note 1, ref. 4).

UNPAVED ROAD INSPECTION REPORT			DATE	15 MAY 1986	INSPECTOR	BELLO
ROAD NO.	B137	SUB-SECTION NO.	4	START km	0+0	SURFACE TYPE	Laterite

		Distance	km 0	25	50	75	+100 0	25	50	75	+200 0	25	50	75	0	ACTION REQUIRED
LEFT	Side drain /turnout	Silt	/	/	/	/	/	/	/	/	/	/	/			Clear drain
		Scour														
	Shoulder	Deform.														
		Scour														
		Vegetation	/	/	/	/	/	/	/	/	/	/	/			Cut grass
Gravel thickness			30 mm													Regravel
RIGHT	Shoulder	Vegetation														
		Scour														
		Deform.														
	Side drain /turnout	Scour														
		Silt	/	/	/	/	/	/	/	/	/	/	/			Cut grass
Notes/comments																
MONITORING	Camber															
	Longtd. Deform.															
	Rutting															
	Corrugations															
	Pot-holes															
Notes/comments																

Figure 21.2 Inspection form for unpaved roads (TRRL Overseas Road Note 1, ref. 4).

Evaluation of the structural condition is usually based on deflection measurements.

Roughness

Roughness is normally measured by a response-type roughness meter which measures the sum of relative displacements between axle and body of a running vehicle. Common instruments are the bump integrator and the Maysmeter which both record the relative displacements between the rear axle and the body of a vehicle. Both can be installed in an ordinary passenger car, although the bump integrator is also available as a one-wheeled trailer.

An accelerometer can also be used as a simple response-type roughness meter. The accelerometer is also installed in a passenger car but the installation only consists of fastening the instrument to the body of the car. The accelerometer measures the vertical acceleration of the body at short time intervals. The results may be recorded in different ways to enable a value of roughness to be determined.

The results from a response-type roughness meter are related not only to the longitudinal profile in the wheel-path of the road surface, but are also dependent on the characteristics of the measuring vehicle. Calibration of the instrument and vehicle are necessary if the results are to be utilized for the calculation of road user costs (ref. 5). Roughness is often recorded on a once-a-year basis for the total road network.

Deflection

Deflection measurements are normally only carried out on road sections where a visual inspection has disclosed possible structural deficiency. The most widely used instrument for deflection measurements is the Benkelman beam. The Benkelman beam is a hand-operated device placed between the twin-wheels of a parked, loaded truck. The truck is then moved away and the deflection of the road surface is recorded.

FWD

The falling weight deflectometer (FWD) is becoming increasingly popular for evaluating the structural conditions of road pavements. Use of the FWD enables the deflection bowl of the pavement under a dynamic load to be measured. The elastic modulus of each pavement layer can then be calculated from the results, provided that the thicknesses of the pavement layers are known (cf. Chapter 16).

Two-stage measurements

Normally the condition survey is carried out in two stages (ref. 6).

- Stage 1: a coarse survey is carried out on the network as a whole to provide network statistics and to identify those sections worthy of further consideration.
- Stage 2: a detailed survey is undertaken of those sections likely to be requiring treatment.

Often, a visual inspection through the windshield of a car supplemented by a roughness survey is used for Stage 1. The visual inspection and the roughness survey are carried out independently. The detailed inspection in Stage 2 is normally undertaken by inspectors walking the road, and often supplemented by deflection measurements. The deflection measurements are carried out on road sections where the visual inspection has disclosed possible structural deficiencies.

Traffic counts and axle load measurements

In order to evaluate the need for traffic-dependent maintenance activities it is important to know the traffic volume and the distribution of axle loads on each section of the road network. Traffic and axle loads may be used to define different road hierarchies between which maintenance standards and thresholds will vary.

Traffic volume

On roads with light traffic, it is normally sufficient to make a rough estimate of the Annual Average Daily Traffic (AADT). On roads with heavy traffic, counts are made at selected count stations with an interval of a few years. Traffic in intervening periods and future traffic are estimated by use of growth factors. Traffic counting and forecasting are outlined in Chapter 3.

Axle loads

The distribution of axle loads is normally measured by means of mobile wheel-weighing scales. Measurements are made at different locations where the composition of the traffic is representative of the different road hierarchies. For example, if the traffic volume is low, all passing vans, trucks and buses are weighed on an ordinary week day. If the traffic volume is large, then a sample is normally taken, for example every tenth passing commercial vehicle is stopped and weighed. Under no circumstances should the survey be limited to vehicles with large, visible loads. The results of the axle load survey are calculated as the average number of equivalent standard axles (ESA) for each category of commercial vehicle (cf. Chapter 16).

Treatment selection

Selection rules

Traditionally, the choice of maintenance treatments in response to defects has been made by the maintenance engineer based on engineering judgement. However, to ensure that treatment is cost-effective and consistent, a more objective approach is appropriate. Treatment selection rules can be formulated which relate maintenance treatments to defect thresholds, or combinations of defects. The selection rules should reflect the maintenance standards. A drawback of the treatment selection based on fixed maintenance standards is that this method does not allow for an evaluation of the consequences if there is insufficient budget to fund all of the work identified as necessary in relation to the standards.

Economic models

A more sophisticated approach considers a number of different alternative maintenance treatments for each link in the road network. One alternative is always 'do nothing'. The maintenance treatments are determined for each road link by economic optimizing (refs 8 and 9). Two different types of economic models are used for optimizing: commercial models and user models.

Commercial models

Commercial models select the maintenance works that are the cheapest solution for the road authority when taking a long-term view. Future maintenance as well as present maintenance are included in the costings. Future maintenance costs are either estimated subjectively, or calculated objectively using submodels that are able to predict the future conditions of the roads. The future costs are discounted back to the present year.

User models

User models include benefits to the road users as well as costs for the road authority during the evaluation period (normally 10 or more years). The benefits to the road users are arrived at in the following way. The road user costs are calculated for different maintenance alternatives. The benefits for a specific alternative are defined as the saving in road user costs for that alternative when compared with the road user costs corresponding to the 'do nothing' alternative. A net present value (NPV) is calculated by subtracting the cost of the maintenance works from the road user cost benefits, as follows:

$$NPV = (RUC_{DN} - RUC_{DS}) - (MC_{DS} - MC_{DN})$$

where:

RUC_{DN} = discounted road user cost in the do nothing case;
RUC_{DS} = discounted road user cost in the do something case;
MC_{DS} = discounted maintenance cost in the do something case;
MC_{DN} = discounted maintenance cost in the do nothing case.

For maintenance to be worthwhile, the NPV must be positive; i.e. the savings in user costs must exceed the costs of maintaining the road. The maintenance need for a road network is defined as the selection of maintenance works that yields the highest NPV for all road links (cf. Chapter 3).

The fact that roads are built for the road users advocates the use of user models rather than commercial models. However, with both methods, the amount of data analysis and calculation required normally mean that computerized methods need to be used to assist with the evaluation. Examples of such methods are included later.

21.2.4 Resource requirements

Having identified the maintenance needs, the resource requirements are calculated in terms of money, personnel, equipment and materials.

Techniques

The resource requirements are related to the maintenance techniques utilized. Machine-intensive methods make heavy demands on foreign exchange. Labour-intensive methods require a massive input of manpower.

Productivity

The resource requirements depend on the productivity levels anticipated from different maintenance activities. The productivity of an activity can vary considerably from country to country, but should lie within the limits indicated in Table 21.1.

It should be noted that the values in Table 21.1 make no allowance for time that is not spent actually working, including non-productive time due to broken down or non-available equipment. Non-productive time can build up significantly during maintenance operations, and it should be an aim of management to reduce it to a minimum.

21.2.5 Priority assessment

Usually funds allocated to maintenance will be insufficient to meet all the identified maintenance needs. This applies to industrialized countries as well as developing countries. An important component of the maintenance management cycle is to prioritize maintenance works to determine which can be carried out in the budget period and which works must be deferred until a later time. Two different types of methods are used for prioritizing: ranking models and economic models.

Ranking models

The simplest method of prioritization is to apply a straightforward ranking to the maintenance works depending on the severity of the defectiveness. A slightly better method is to relate priority to the importance of the different road links and partly to the importance of the different maintenance works, rather than defectiveness. In such an approach, main roads are given higher priority than feeder roads, and cleaning and reshaping ditches may be given higher priority than resealing pavements. The resulting works are placed in a list in order of their rankings and

Table 21.1 Outputs of work (TRRL Overseas Road Note 1, ref. 4)

Activity	Resource requirements			Output unit	Range of outputs	Remarks
	Men	Equipment	Materials			
Clearing side drains – hand	4-10	Shovels, cutlasses, picks	—	m/man-day	30-60	
Clearing side drains – machine	2-3	Grader, shovels	—	km/day	4-7	
Re-excavating side drains	2-10	Picks, shovels	—	m/man-day	8-15	Output depends on type of material.
Clearing culverts	2-4	Shovels, headpans/ wheel-barrows	—	no/man-week	2-4	
Minor repairs to culverts	2-4	Masons tools	Cement, aggregate, sand	no/man-week	2-10	
Major repairs to culverts	To be assessed for each job			man-day.		To be fixed for each job.
Making culvert rings (1m diameter × 1m long)	4-10	Moulds, mixer, shovels	Cement, stone, sand, reinforcement.	no/day	5-10	Made in works yard. Stand-by job for concrete team.
Grading unpaved surfaces	2	Grader, camber-board, spirit-level	—	pass-km/ day*	20-50	
Dragging unpaved surfaces	1	Tractor and drag	—	pass-km/ day*	20-50	Up to 5 passes may be needed to achieve satisfactory results.
Patching bituminous surfaces	5-7	¼ tonne roller or hand-rammers, brushes, picks, shovels, watering-cans.	Premix or laterite gravel, bitumen emulsion, chippings or washed gravel.	m^3/man-day	0.5-0.8	
Filling gravel surfaces	5-7	¼ tonne roller or hand-rammers, brushes, picks, shovels, watering-cans.	Gravel	m^3/man-day	0.6-1.2	
Filling earth surfaces and slopes	4-5	Hand-rammers, brushes, picks, shovels.	Selected earth.	m^3/man-day	0.9-1.5	

* The unit of 'pass-km' is the actual distance the grader travels while working. To determine the length of road graded, this figure must be divided by the number of 'passes' necessary to cover the whole width of the road.

Table 21.1 *Continued*

Activity	Resource requirements			Output unit	Range of outputs	Remarks
	Men	Equipment	Materials			
Grass cutting – hand	2-10	Cutlasses	—	m²/man-day	300-800	
Grass cutting – machine	1-2	Tractor/ mower.	—	km/day	10-20	Single pass machine
Repairing and replacing traffic signs	2-3	Masons' tools, painters' tools, shovels.	Cement, stone, sand, paint, reflective paint.	no/man-day	4-8	
Road markings	2-4	—	Road paint	m/man-day	50-200 (hand painting)	Machine painting will achieve up to 10km per machine.
Stockpiling gravel – hand	10-20	Picks, shovels.	—	m³/man-day	0.5-2.0	
Stockpiling gravel – machine	4	Bulldozer/ mechanical shovel	—	m³/day	450-500	
Regravelling gravel surfaces	12-20	1 grader, 8 tippers, 1 loader, 1-2 water tankers, 1 roller.	Gravel	m³/day	300-350	Assumes own labour for supply of gravel. If gravel is supplied by contract, omit tippers and loader.
Surface dressing	15-20	1 distributor, 1 roller, 3 tippers, 1 gritter, 1 loader.	Bitumen, chippings	lane-km/day	2.5-4.0	
	+ 10 additional if no loader available					
	+ 10 additional if no gritter					

cumulative costs are applied, starting at the top of the list. Maintenance works with the lowest rankings are rejected until the total costs of the remaining works equals the reduced budget. The main problem with ranking models is that they do not take into account the longer-term consequences of decisions, with the result that constrained budgets are not spent in the optimum manner.

Economic models

The more sophisticated systems adjust the list of repair works by use of economic and life cycle cost considerations. For each section of road, a range of possible treatments is considered. The approach selects the best combination of treatments across the network as a whole to optimize the use of the available budget. A consequence is that, instead of rejecting expensive maintenance operations, these are replaced by cheaper solutions. For example, resealing may be substituted by patching; overlay may be substituted by resealing. In this way, the maintenance can be stretched to a larger part of the road network and a better total solution is obtained.

Commercial models

The criterion used by commercial models is that the selection of maintenance treatments should result in the lowest possible sum of the future maintenance

costs discounted back to the present year. Maintenance solutions that are cheaper than the optimum solutions will, inevitably, increase the future maintenance needs and costs, but a commercial model minimizes the total cost for the road agency.

User models

User models operate in a similar manner to commercial models, but also include road user costs. When consideration is given to substituting optimum maintenance solutions by cheaper solutions, the sum of the costs includes both future maintenance costs and road user costs. A user model selects treatments to minimize this future cost.

The calculations are rather complicated for both commercial and user models. This is because the road network normally consists of many links and several alternative maintenance treatments must be considered for each link. The number of combinations of different treatments applied to different links becomes extremely high. Taking a road network with 50 links as an example: if the number of alternative maintenance works on each link is five, then the number of combinations is 5^{50}. Added to this is the complication that, for each combination, the future costs must be calculated year by year and then discounted back to the present year. Different methods are used to reduce the number of combinations but, even then, electronic data processing is needed to handle the calculations.

Consequence analysis

An important additional benefit of the economic models is that they can be used for consequence analysis. For different budgets, the future condition of the roads, maintenance needs, and road user costs can be evaluated. This information is particularly valuable for decision-makers at all levels.

21.2.6 Scheduling and execution of works

The next task is to prepare detailed work schedules for the maintenance teams. The schedules are essentially a set of instructions which tell the foremen or technicians supervising an activity how much work is to be done each day, the time it should take and the labour, equipment and materials to be used.

Worksheets

Figure 21.3 shows an example of a completed worksheet. The target is the first item entered on the worksheet. The amount of each resource to be used is entered next in the top of each line. The worksheet is issued to the supervisor who enters in the bottom half of each line the progress actually made and the resources used day by day. At the end of the schedule period, the completed form is returned to the maintenance engineer to calculate how much of the production target has been achieved.

21.2.7 Monitoring

Monitoring serves two main purposes: to check the quantity and the quality of the work being done and to provide data that can be used to improve future maintenance operations. Monitoring involves site visits and desk review.

Site visits

Site visits are important because they enable the maintenance engineer to become thoroughly familiar with road conditions in the area and to gain a first-hand knowledge of the extent and the quality of the maintenance that has actually been carried out. The presence of the maintenance engineer on the spot means that he can advise on problems as they arise, and seeing him regularly on the site

WORKSHEET	PERIOD	5-18 Oct 86			ROAD NO.	C 231			SUB-SECTION NO.	2			ACTIVITY	Regravelling				
	Unit	M	T	W	T	F	S	S	M	T	W	T	F	S	S	Totals	%
Target production	km	—	—	0.5 km/day					—	—	—	—	—	—	—	7/5	71
Resources																		
Gravel (Subcontract)	m^3	—	—	350 m^3/day					—	—	—	—	—	—	—	4900	83
		300	300	300	100	400	400	300	250	300	300	100	300	400	300	4050	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	
Grader	h	—	—	8/day				—	—	—	—	—	—	—	—	112	77
		6	8	8	4	8	b/d	8	8	8	4	8	8	8	b/d	86	
Roller (2)	h	—	—	16/day				—	—	—	—	—	—	—	—	224	74
		16	16	7	8	16	—	16	16	16	16	16	8	16	—	167	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	
		—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	

Figure 21.3 Example of worksheet (TRRL Overseas Road Note 1, ref. 4).

should boost the morale of road gangs and improve their standard of work and their output.

Desk review

Desk review is an office task that involves reviewing all the maintenance documentation, i.e. inspection reports, completed worksheets, etc. It provides the opportunity to assess the performance of the maintenance programme and the effectiveness of the management system.

21.3 MAINTENANCE MANAGEMENT SYSTEMS

Use of computers

It is clear that, in order to assess maintenance needs and to allocate budgets, data must be collected about the road network and its condition. The volumes of data needed, even for small road networks, point to computers as being a rational choice for carrying out the analysis associated with maintenance management. Several commercial software packages are available to assist with this task. Systems are available for managing pavements, bridges and structures, routine maintenance, roadside and safety features, equipment, and maintenance operations. Systems for managing the last item are sometimes, rather confusingly, called simply 'maintenance management systems'. Emphasis in the remainder of this chapter is on 'pavement management systems' – often summarized as PMS.

Management systems have been implemented in a number of developing countries during recent years. However, in some cases, the systems have proved to be ineffective (ref. 2). If the introduction of a system is to lead to real improvements in management performance, experience shows that the following conditions should be met:

- the system must be adapted to suit the local conditions;
- existing methods and procedures that cannot be used unchanged should be modified rather than replaced with new ones;
- sufficient, well-trained and motivated staff must be provided;
- the introduction of the system should be properly supervised.

Adaptation

A system copied unchanged from another country is unlikely to be successful. Pavement types, methods of measuring pavement distress, unit costs, methods for scheduling work, etc., all differ from country to country. The system should be adapted to the local conditions, and the adaptation should be made in close co-operation with the local authorities.

Modification

As far as possible, management systems should be modified to utilize existing methods and procedures, although some modification of local practices is likely to be needed. Where changes to local practices are necessary, they should be introduced gradually, rather than being rejected and replaced by completely new methods and procedures. Evolution is always met with less resistance than revolution.

Staff

Sufficient qualified staff must be provided to operate the system. The staff must be trained, and the training should be a continuing activity to ensure that new staff can also acquire the necessary skills. The staff must be motivated and made to understand that the management system will be an advantage both to themselves and to the organization. Training is discussed in Chapter 25.

Supervision

It will normally be necessary for the local road organization to be assisted, at all

levels, by experienced consultants during introduction of a system. The assistance should continue until the staff is totally familiar with all aspects of the system operation.

A few different pavement management systems are outlined briefly in the following sections. The focus is on the main components: (a) assessment of maintenance needs, and (b) priority assessment. In the next section, a manual system is described and, in the last section, examples of computer-based systems are given.

21.4 EXAMPLE OF A MANUAL SYSTEM

Overseas Road Note 1 (ref. 4) is published by the Transport Research Laboratory (TRL) in the United Kingdom. The note is a practical guide to the management of maintenance operations. The note was first published in 1981 and may now be a little outdated. However, the guide is an eminent example of a simple, down-to-earth system. In many cases, a simple system like this has a better chance of sustainability than a system based on sophisticated hardware and software, and on staff with special skills.

Needs

Maintenance needs are assessed by use of maintenance standards and treatment selection rules. Table 21.2 shows an example of treatment selection rules for paved roads.

The condition survey is based on visual inspections. If inspection identifies defects that may be of a structural nature, then further investigations are required. As seen from Table 21.2, deep rutting and extensive cracking call for further investigation. Further investigation may include objective measurement of pavement strength using techniques such as deflection measurements.

Priority

Priority assessment is based on a simple ranking model. Priorities are set depending partly on the importance of the maintenance activities and partly on the importance of the road. Maintenance activities may be ranked in order of importance, and Overseas Road Note 1 suggests rules for this.

An example of road hierarchy (category), based on traffic and surface type, is shown in Table 21.3.

Finally, Table 21.4 shows how the priority of maintenance treatments for different road types is determined. Maintenance activities are numbered from 1 (highest priority – urgent work on strategic roads) to 48 (lowest priority – special works on unpaved roads with very low levels of traffic). The matrix ensures that every road in the network receives at least the minimum maintenance needed to keep it operational, while at the same time focusing recurrent and periodic maintenance on the economically important roads with high traffic levels.

21.5 EXAMPLES OF COMPUTER-BASED SYSTEMS

Pavement management systems have been implemented extensively in Europe, North America and Australasia, as well as in several developing countries. Although the essential components of the systems are the same in each case, demands

Table 21.2 Treatment selection rules for paved roads (TRRL Overseas Road Note 1, ref. 4).

Defect	Level	Extent (% of sub-section length)	Climate/traffic category	Defect	Extent (% of sub-section length)	Action	Programme	Notes
Stripping or fretting	Any	< 10	All	—		Local sealing	Recurrent	A fog spray of emulsion may be sufficient to renew the surface
		> 20	All	—		Surface dress	Periodic	
Fatting-up or bleeding	—	—	All	—		No action	—	Local sealing or surface dressing may be required if the lack of skid resistance is a problem. In this case, the excess binder must be burned off first. Sanding is appropriate when live (shiny) bitumen is on the surface.
Pot-holes	Any	—	All	—		Patch	Recurrent	Extensive pot-holing may result from lack of effective maintenance or rapid deterioration of the road structure or surfacing. The cause must be determined and appropriate action taken.
Edge damage	Erosion from original edge > 150mm	> 20	All	—		Patch road edge and repair shoulder	Recurrent	If the failure is severe or persists, reconstruct the shoulder.
Edge step	> 50mm	> 50	All	—		Reconstruct shoulder	Period	
Wheeltrack rutting (surface dressing on granular base)	< 10mm	—	Rainfall > 1500mm/yr OR Traffic > 1000 vpd	Wheeltrack cracking	< 5	Seal cracks	Recurrent	
					> 5	Surface dress	Periodic	Single seals are often insufficient for wide cracks.
				Non-wheeltrack cracking	< 10	Seal cracks	Recurrent	
					> 10	Surface dress	Periodic	See note above.
			Rainfall < 1500mm/yr AND Traffic < 1000vpd	Wheeltrack cracking	< 10	Seal cracks	Recurrent	
					> 10	Surface dress	Periodic	See note above.
				Non-wheeltrack cracking	< 20	None	—	
					> 20	Surface dress	Periodic	See note above.
	10-15mm	> 10	All	Any cracking	—	Treat cracks depending on extent as above.	Recurrent/ Periodic	If rate of change of rut depth is slow.
						Further investigation	—	If rate of change of rut depth is fast.
	> 15mm	< 10	All	Cracking only associated with local ruts	—	Patch	Recurrent	
				Other cracking	—	Patch excess rutting and treat cracks depending on extent as above	Recurrent/ Periodic	
		> 10	All	Any cracking	—	Further investigation	—	

Table 21.2 *Continued*

Defect	Level	Extent (% of sub-section length)	Climate/traffic category	Defect	Extent (% of sub-section length)	Action	Programme	Notes
Wheeltrack rutting (asphaltic concrete on granular base)	< 10mm	–	Rainfall > 1500mm/yr OR Traffic > 1000 vpd	Any cracking	< 5	Seal cracks	Recurrent	
					5-10	Surface dress	Periodic	Single seals are often insufficient for wide cracks.
					> 10	Further investigation	–	
			Rainfall < 1500mm/yr AND Traffic < 1000 vpd	Any cracking	< 10	Seal cracks	Recurrent	
					10-20	Surface dress	Periodic	See note above.
					> 20	Further investigation	–	
	> 10mm	< 5	All	Cracking only associated with local ruts	–	Patch	Recurrent	
				Other cracking	–	Patch excess rutting and treat cracks depending on extent as above	Recurrent/ Periodic	
		> 5	All	Any cracking	–	Treat cracks depending on extent as above	Recurrent/ Periodic	If rate of change of rut depth is slow.
						Further investigation	–	If rate of change of rut depth is fast.
Wheeltrack rutting (asphaltic concrete or surface dressing on stabilised road base)	< 5mm	–	Rainfall > 1500mm/yr OR Traffic > 1000 vpd	Any cracking	< 10	Seal cracks	Recurrent	Includes reflection
					> 10	Seal cracks and surface dress	Periodic	
			Rainfall < 1500mm/yr OR Traffic < 1000 vpd	Any cracking	< 20	Seal cracks	Recurrent	
					> 20	Seal cracks and surface dress	Periodic	
	5-10mm	> 10	All	Any cracking	–	Treat cracks depending on extent as above	Recurrent/ Periodic	If rate of change of rut depth is slow.
						Further investigation	–	If rate of change of rut depth is fast.
	> 10mm	< 5	All	Cracking only associated with local ruts	–	Patch	Recurrent	
				Other cracking	–	Patch excess rutting and treat cracks depending on extent as above.	Recurrent/ Periodic	
		> 5	All	Any cracking	–	Further investigation	–	

Table 21.3 Road hierarchy (category), based on traffic and surface type (TRRL Overseas Road Note 1, ref. 4).

Category	*AADT*	*Surface type*
1	(Strategic roads)	Paved
2	Greater than 1000	Paved
3	500–1000	Paved
4	200–500	Paved
5	Greater than 200	Unpaved
6	Less than 200	Paved
7	50–200	Unpaved
8	Less than 50	Unpaved

Table 21.4 Matrix of maintenance priorities (TRRL Overseas Road Note 1, ref. 4).

Category of maintenance activity	*Priority*							
	Road hierarchy							
	1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
Urgent work	1	7	8	9	10	11	12	13
Routine drainage work	2	14	15	16	17	18	19	20
Routine work on pavement	3	21	24	27	30	33	36	39
Periodic work (regravelling, surface dressing	4	22	25	28	31	34	37	40
Other routine work	5	23	26	29	32	35	38	41
Periodic work (overlaying, reconstruction)	6	42	43	44	45	46	47	48

placed on a PMS will differ because of the different levels of national development. Examples of these are given in Table 21.5.

These factors suggest that the requirements for an appropriate PMS will be different in countries at different levels of national development. For example, the inclusion of roughness-related vehicle operating costs should be essential for developing country PMSs, whereas some industrialized countries consider that this is unnecessary. Congestion effects on vehicle delay and time costs, as a result of high traffic volumes, should be included in industrialized countries' systems, whereas developing-country systems may need to consider congestion as a result of high proportions of slow moving vehicles in the traffic stream. Different physical system designs are necessary for the two environments, targeted at the different sophistication of management methods and the different general levels of computer literacy.

Table 21.5 Differing requirements of PMSs.

	Industrialized countries	*Developing countries*
Development of management methods in maintenance organizations	High	Low
Computer literacy of maintenance engineers	Moderate to high	Low
Data collection methods in use	Extensive use of visual and high-speed mechanized methods	Limited use of visual and high-speed methods
General pavement conditions	Relatively smooth surfaces	Generally rough surfaces
Traffic densities	Generally high to congested	Relatively low
Vehicle mix	Dominated by private cars	High proportions of commercial vehicles and non-motorized transport
Dependence on foreign aid	Low	High

PMSs operating in industrialized countries will, therefore, not be discussed further, but the following section cites some examples of PMSs operating in some developing countries.

Reservations

Some PMSs have been developed using submodels from the World Bank HDM-III model to forecast the future deterioration of the road pavement. However, this use of the HDM model is highly debatable.

The HDM deterioration models are derived from field observations and quite naturally most thick pavements carry heavy traffic and many thin pavements carry light traffic. As a consequence the derived pavement deterioration correlates unrealistically weakly with traffic loading and pavement thickness. It looks like the deterioration is not very dependent on the traffic loads. This is the main reason why the HDM submodels are not suited for proper pavement design and detailed calculation of the pavement response to traffic. Detailed calculation of the pavement response is fundamental in a PMS.

dROAD/dTIMS

The approach to PMS adopted by Deighton Associates in implementations in Botswana, India, Indonesia, Lesotho, Trinidad and Thailand contained three main elements:

- a database and associated systems and procedures for its maintenance (dROAD);
- a PMS module (dTIMS);
- an automated mapping system that allows access and display of the database information on a map (dMAP).

The dROAD database system provides the core of the system storing all network, inventory and condition data. The road deterioration and maintenance effects prediction submodel from the World Bank's HDM-III, or any set of user-defined deterioration models, can be specified in conjunction with user-defined user-cost relationships to set economic network-level standards, and dTIMS has

facilities for incorporating these. dTIMS is designed to function in conjunction with the dROAD database and provides a PMS engine for treatment selection, estimating, priority ranking and the preparation of budgets. dTIMS is parameter-driven and requires customizing to individual implementation requirements. Data collection in the system is customized to each implementation.

The three modules of the system comprise all of the primary elements of a PMS including: setting economic maintenance standards for the network, managing data, and providing a works programme and budget.

IN-ROADS

IN-ROADS has the specific objective of providing a system for municipal and island authorities with limited resources for maintenance management. The system is based on very simple data requirements, but uses economic models for consequence analysis and to assist in setting priorities under conditions of budget constraint. The system was developed by May Associates from work undertaken in Zanzibar, where a version of the system is now fully operational.

A simple register is set up using basic data about the road network. Condition is assessed by rating carriageway defects on a scale of 1 to 5, and up to five non-carriageway items each on a scale of 1 to 3. Treatment selection rules are simplified, but can include widening of bitumen roads and upgrading of gravel roads to bituminous standard. Treatment costs are assigned on a per-kilometre basis for routine works varying with condition, traffic, etc. Periodic treatments and rehabilitation/reconstruction works are costed and prioritized on a user-defined basis which takes into account condition, traffic and the economic function of the road.

A facility is provided for undertaking consequence analysis of future condition based on different budget availabilities using simple deterioration models, designed for easy customizing by the user. The consequence analysis can be used to demonstrate the effect, in condition terms, of not allocating money at the rates requested, and provides a basis for setting user-defined prioritization functions. Outputs are provided in tabular and graphical form for work planning and budget preparation purposes.

NETTER-PMS

NETTER-PMS has been developed by Rendel Palmer & Tritton and has been implemented in India and Malaysia.

It contains identical pavement deterioration and vehicle operating cost relationships to HDM, but it is structured differently from HDM and completely reprogrammed to avoid any restrictions on the number of links which can be dealt with in a single run. This module is used as a PMS 'engine', in lieu of HDM-III. This economic model allows the choice of maintenance treatments to be optimized across the network as a whole, using future cost considerations, when budgets are constrained. NETTER-PMS also differs from HDM by including speed–flow relationships to handle congestion effects and models user delays due to maintenance works.

NETTER-PMS operates in conjunction with a relational database and contains the following:

- NETTER PMS 'engine';
- integral interface between NETTER and the PMS database;
- input of network, inventory and condition data through menus, or directly from electronic data capture devices;
- text-based and graphical reporting.

The system can be used at both network and project level, and is designed to operate, if required, in a decentralized organization. Policy data are entered at headquarters, whilst condition data, such as roughness, deflection and visual condition data, are entered locally. This enables local priority programmes of works to be developed and then submitted to headquarters for possible funding.

PEMM

The Pavement Evaluation and Maintenance Management System (PEMM) was developed to carry out feasibility studies at project and network level by the Carl Bro group of companies. The system has been implemented in Chile and Malawi.

It provides a user-friendly data input and preprocessing facility to HDM-III, containing:

- a data generation and management subsystem;
- a technical information generating subsystem;
- an interface to HDM-III;
- an impact analysis and maintenance programming module.

Data collection is carried out following standard procedures and makes use of statistical sampling processes to differentiate between network and project level data. Data are recorded on special forms. The data are stored in a separate database which has menu-driven input and output routines.

The technical information subsystem retrieves data from the database and provides graphics facilities to assist the engineer to work interactively to divide the road data into sections of approximate uniform structural and functional performance, and to produce the information required for running HDM-III. Assistance is given with the preliminary design of overlay and reconstruction projects.

An interface module converts this information into standard input files to enable HDM-III to be run, and a further program can be used to provide graphical output of the run results. In its simple use, HDM-III can only set priorities with an unconstrained budget. HDM-III can deal with a constrained budget using an expenditure budgeting model, but most people find this model cumbersome and do not use it.

The impact analysis and maintenance programming module provides the facilities to obtain the optimum network maintenance plan for a user-selected rolling budget.

SMEC

The SMEC system was developed by the Snowy Mountains Engineering Corporation, an Australian-based international engineering consultant. It has been implemented in Hong Kong and the Philippines.

The system is based on HDM-III which is fully integrated with a database. It can be operated at both network and project level; the same data, operating systems and optimization processes being used at each level. The principles of the system are as follows:

- pavement structure, condition, traffic, cost and related data are loaded into the relational database;
- the data are either entered on the basis of homogeneous sections, or used to subdivide the road into homogeneous sections for analysis by HDM-III;
- reports can be produced on present conditions, future conditions, treatment costs and the like, as required for sections or links on a road, regional or state basis;

- HDM-III is used to predict the following for each road section:
  (a) deterioration in terms of roughness, pot-holes, cracking, rutting and ravelling under defined routine maintenance options;
  (b) vehicle operating costs and costs of undertaking routine maintenance on each road section under defined routine maintenance options;
  (c) the deterioration of each road section if different upkeep and improvement works are carried out at different times in the future under different maintenance scenarios;
  (d) vehicle operating costs and the cost of undertaking routine maintenance on each road section under different maintenance and improvement policies.

HDM-III is then used to run optimization and consequence analysis using different budgets, routine maintenance programmes, treatment options (including 'must-do' projects), costs and the like to assist in making decisions on road maintenance programmes. Constrained budgets may be dealt with using the expenditure budgeting model.

BSM

The Burrow Snaith Maintenance (BSM) system was developed by Highway Management Services and includes five 'integrated packages':

- an inventory database for roads of all surface types, to handle the selection of renewal and improvement projects for bituminous surfaced pavements in priority order;
- a complementary analysis, based on engineering and economic criteria, which interrogates the database, subsection by subsection, to determine the appropriate remedial treatment where one is required, and to place that into a priority order of works using a simple ranking method; future deterioration is not predicted;
- an interface to HDM-III for carrying out an economic analysis of proposed renewal and improvement projects, thereby reviewing the priority works list;
- an optional traffic statistics database which provided traffic census and supplementary data for the other three packages;
- auditing routines to determine the overall condition of designated roads or road networks, together with changes that may have occurred with time; condition and treatment history are available within this package.

The BMS system includes all required PMS processes for bituminous roads at network and intermediate operational levels, and includes project identification at project level. The original program suite was developed several years ago and has been upgraded progressively following a series of implementations in Botswana, China, Cyprus, India, Malaysia and Thailand.

The road network is divided into fixed manageable subsections for which inventory information is recorded by a field survey team. Depending on available resources, one way of operating a stepped data collection approach is to conduct a high-speed condition survey, usually measuring roughness in concert with a drive-over visual assessment, to identify subsections likely to require treatment. A more detailed pedestrian and objective survey, using for example the falling weight deflectometer, may then be carried out by the field survey team and these data are then added to the system's condition database.

BSM uses these data to determine appropriate treatments using intervention levels, which have been predetermined on the basis of the results of multiple runs

of HDM-III, and engineering judgement. A multi-criteria priority algorithm of condition and traffic is then used to determine which of these projects are likely to be funded. HDM-III runs are then prepared for these potential projects using a menu-driven interface program that compiles the input data. The HDM program is then run from the main menu. The results of these runs are fed back into the BSM program for budget allocation and the identification of projects for design.

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PART 9

Development Assistance



Road project in Vietnam assisted by EC. (Photo by Heine Pedersen)

22 Development aid

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22.1 INTRODUCTION

End of colonial rule

The Second World War became in many ways the beginning of a new era for what is today commonly referred to as the Third World countries. Many of those countries supplied soldiers to the allied nations during the war and through the war effort obtained a new conception of their own worth. In many countries old independence movements got a push forward during the war years and the end of the war marked the beginning of the end of the old colonial rule.

One of the first colonies to obtain independence was India in 1947. In the following years freedom movements expanded in most of the old European dependencies in Africa and Asia. Mention may be made of Algeria, Kenya and Indonesia where the struggle for independence quickly gathered momentum and was accompanied by the development of guerrilla-type insurrection.

Continued co-operation

As part of the phasing out of colonial rule most of the old colonial powers continued to co-operate with their former colonies. Such co-operation has mostly continued to date, for example, the political and economic co-operation within the Commonwealth countries of former British dependencies. It soon became clear that the need for co-operation with and assistance to the developing countries far exceeded the capacity of the old colonial powers. Gradually an increasing number of industrialized countries began to include development aid as part of their foreign relations.

Aid concept

Development aid in the form known today started to take effect in the beginning of the 1960s. The concept of development aid has changed over time and the United Nations, especially through their subsidiary the World Bank, has played a leading role in developing aid concepts and in streamlining procedures for preparation and monitoring development aid activities. In recent years this can especially be seen in relation to the economic reform programmes being prepared

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and implemented in many developing countries under guidance and assistance from the World Bank and the International Monetary Fund.

Reasons for providing aid

There are several reasons why countries provide development aid. An important reason for most donor countries is commercial considerations. Through assistance to the developing countries the donor countries attempt to increase trade and enhance common economic growth. During the cold war, geo-political issues were also to a large extent what drove the super powers to facilitate aid schemes. Today idealistic and humanitarian reasons play an increasing role reinforced by the presence of advanced communication and information technology. The developed world knows what the conditions are, almost on a day-to-day basis in the developing countries through the media and people in the Third World also know about the conditions in the industrialized countries. This has resulted in a growing feeling of interdependency.

Also of late, security considerations form part of the participation for using funds (taxpayers' money) on development co-operation. The reasoning is that only through improved living conditions and economic and social progress can political stability be achieved. This is considered a prerogative for international security.

In recent years, environmental issues have become a significant area for development co-operation. It is realized by all that environmental issues are regional or global, not national or local. The industrialized countries should help the developing countries to avoid at least some of the mistakes which the industrialized countries have made in the past.

22.2 RESOURCE TRANSFERS

Financing of road projects

By its nature road construction is predominantly a public undertaking by a government ministry/department or by a local authority such as a provincial government or council. It is not least because of this that it is necessary for professionals who work with roads and traffic in the developing countries to be conversant with the principles of developing co-operation. Donors' financing of projects play an important role in many developing countries. This is illustrated by Table 22.1 which shows official development assistance as percentages of GNP for some selected countries. (Official development assistance or aid is defined as resource

Table 22.1 Official development assistance as percentages of GNP for selected countries 1991 (ref. 1).

Country	*% of GNP*
Mozambique	69
Tanzania	34
Zimbabwe	6
Nicaragua	48
Bangladesh	7
Bhutan	25

Table 22.2 Total net resource flows from DAC member states to the developing countries 1992 (ref. 2).

	US$		
Type of transfer	*Billion*		*%*
Official development assistance:			
Bilateral	32		
Bilateral loans	9		
Multi-lateral	20	61	57
Other official flows:			
Bilateral	7		
Multi-lateral	1	8	8
Private flows:			
Direct investment	23		
Bilateral portfolio	12		
Multi-lateral portfolio	−2		
Export credits	−1	32	30
Grants by private agencies	5	5	5
Total net flow		106	100

flows that are concessional in character and convey a grant element of at least 25%).

Multi-lateral/ bilateral aid

Development aid is normally divided into multi-lateral assistance and bilateral assistance respectively. Multi-lateral assistance is aid transferred from a donor country via international aid organizations. Bilateral assistance is aid that the donor gives in direct co-operation with the recipient country.

Assistance from DAC countries

Prior to 1989 the Eastern Bloc provided a sizeable share of the transfers from industrialized to developing countries. However, following the breakdown of the Soviet Union the transfers from the countries belonging to the Organization for Economic Co-operation and Development (OECD) dominate the picture. Thus total net transfers from the members of the OECD's Development Assistance Committee (DAC) in 1992 were US$106 billion comprising both official development assistance and private transfers. About 66% of the total net transfer was in non-debt creating conditions. For countries outside DAC, official development assistance was considerably less (US$4.2 billion in 1991).

In Table 22.2 the economic transfers from DAC member states are divided into different types. It appears that official development assistance amounted to a total of US$61 billion or 57% of the total net flows. Bilateral transfer of US$48 billion dominated over US$21 billion multi-lateral transfer. Private transfers including private investments, bank and bond loans and assistance from Non-Government Organizations (NGOs) amounted to US$37 billion.

Details of official transfer

Table 22.3 details the official development aid from the DAC countries in 1992. In 1970 the UN set as a target that international development aid should reach 0.7% of the Gross National Income (GNI). Twenty-four years after this recommendation the aid from the DAC countries only amounts to 0.33% of the total

Table 22.3 Official development aid from the DAC countries 1992 (ref. 2).

Country	*Millions of US$*	*% of GNI*	*Bilateral (%)*	*Multi-lateral (%)*
Norway	1273	1.16	64	36
Sweden	2460	1.03	72	28
Denmark	1392	1.02	54	46
Holland	2753	0.86	68	32
France	8270	0.63	76	24
Finland	644	0.62	65	35
Canada	2515	0.46	68	32
Switzerland	1139	0.46	59	41
Germany	7572	0.39	69	31
Belgium	865	0.39	63	37
Portugal	302	0.36	85	15
Australia	973	0.35	76	24
Great Britain	3217	0.31	53	47
Italy	4122	0.34	59	41
Japan	11151	0.30	75	25
Austria	556	0.30	76	24
Spain	1518	0.26	72	28
New Zealand	97	0.26	76	24
USA	11709	0.20	67	33
Ireland	69	0.16	41	59
Total	60384	0.33	68	32

GNI. Only four countries have reached and passed the target. USA is the biggest single donor but taken as a proportion of the GNI, USA is contributing less than all other countries except Ireland.

The ratio of bilateral aid to multi-lateral aid varies. On average 32% of the aid was multi-lateral but the biggest European donor, France, only contributed with 24% of the aid as multi-lateral.

Future aid level

Future aid levels and characteristics are likely to undergo changes as policies and priorities change over time. In recent years increased attention has been given by donor countries to issues of democracy and demand driven economies, basis human rights and environmental problems. Donor countries are likely to utilize development co-operation as a vehicle for improvement of respect for basic human rights and democratic reform.

The high priority being accorded co-operation between donor nations and the former Eastern Bloc countries also means that resources are being diverted away from potential recipients in traditional Third World countries to new co-operating partners. Increased competition for available funds is a likely scenario for several years to come. Adding up to the competition is the fact that several donor countries are reducing their total future aid volume, e.g. Sweden and Finland because of domestic economic problems.

Many developing countries carry very large debt burdens and although many donor nations write off their old government loans this remains a major issue. Several developing countries have negative net resource inflows because of heavy debt repayments.

22.3 INTERNATIONAL AID AGENCIES

Multi-lateral aid is mainly transferred to the developing countries through the World Bank Group, the Regional Development Banks, the UN system and the European Union.

Membership of international aid organizations comprises both developed and developing countries with each country contributing to the operation of the organization. Members' contributions generally reflect their economic strength.

22.3.1 Financial institutions

World Bank Group

The most important multi-lateral financial institution is the World Bank Group. It includes the International Bank for Reconstruction and Development (IBRD), the International Development Association (IDA), the International Finance Corporation (IFC), the International Monetary Fund (IMF), and the Multi-lateral Investment Guarantee Agency (MIGA), all with head office in Washington DC. Practically all countries are members of the IBRD and IDA.

Objective

The objective of IBRD is to provide financial assistance on commercial terms. IDA provides long-term interest-free credits which carry a nominal service charge only. IDA credits are only available to the poorer countries with a Gross National Product (GNP) per capita of less than US$675 (1992). The IFC provides financial assistance for the development of the private sector. The IMF provides credits in order to improve balance of payments and supports monetary stability. MIGA has as its objective to encourage foreign direct investment in developing countries by protecting investors from non-commercial risks, especially risk of war or repatriation. The World Bank Group works worldwide.

Regional banks

The objective of the Inter-American Development Bank (IDB) with head office in Washington DC is to support economic development in Latin America. The African Development Bank (AfDB) and its development fund (AfDF), both with head office in Abidjan, Ivory Coast, are operating in Africa. The Asian Development Bank (AsDB) and its development fund (AsDF) with head office in Manila, the Philippines, operate throughout Asia. Membership of the regional banks is normally by members of the OECD plus countries in the region concerned.

IFAD

The International Fund for Agricultural Development (IFAD) with head office in Rome is a special UN fund for financing investment in the agricultural sector in developing countries.

The net amount of funds transferred to the developing countries from international financial institutions in 1992 is listed in Table 22.4.

The financial institutions, particularly the World Bank, contribute to policy change through structural adjustment lending and financial support to programmes and projects.

22.3.2 The UN system

The United Nations form an umbrella for more than 50 different specialized agencies. Some of these have activities in all member states including the developed countries. Others are pure development aid agencies with activities in the developing countries and some former Eastern Bloc countries only.

Table 22.4 Net disbursements from financial institutions 1992 (ref. 2).

	Millions of US$	
Institution	*Concessional*	*Non-concessional*
IBRD		−808
IDA	4822	
IFC (1991)		1385
IMF	734	
IDB	73	838
AfDB		1177
AfDF	678	
AsDB		1352
AsDF (1991)	1058	
IFAD (1991)	117	
Others	25	189
Total (estimated)	7507	4222

The UN assistance is generally in the form of grants and technical assistance, and training-specialized agencies implement both projects and programmes within their subject area.

UNDP

United Nations Development Programme (UNDP) is the central body for financing and co-ordinating the UN's development aid. 20% of the UN's development aid was channelled through UNDP in 1992.

Projects financed through UNDP are normally implemented by specialized UN agencies. In 1992 the biggest implementing agencies were the Food and Agriculture Organization (FAO), the Department of Technical Co-operation (DTCD), and the International Labour Office (ILO). ILO is interesting from a highway point of view because ILO is very active in promoting labour-intensive methods in road construction and maintenance.

Other UN agencies

The World Food Programme (WFP) is the largest UN organization in money terms. WFP provides food assistance to developing countries. More than half of the projects implemented are so-called 'food for work' projects, many of which include road construction and maintenance. WFP also distributes food as emergency relief to refugees. The United Nations High Commissioner for Refugees (UNHCR) is in charge of international aid to refugees. The aid from this agency includes emergency relief, development projects and education.

The objective of the United Nations Children's Fund (UNICEF) is to improve the condition of life for children and mothers. The effort is concentrated on health, nutrition, water, sanitation and education. The United Nations Transition Assistance Group (UNTAG) has assisted Namibia and Cambodia in the transition to independence and democratic elections.

The United Nations Relief and Works Agency (UNRWA) is a special aid organization for Palestinian refugees. The United Nations Fund for Population Activities (UNFPA) promotes family planning activities. The organization mainly

Table 22.5 Net disbursements from UN agencies in 1992 (ref. 2).

Organization	*Millions of US$*
WFP	1575
UNDP	1164
UNHCR	1068
UNRWA	744
UNICEF	308
UNTAG	242
UNFPA	126
Other UN agencies	659
Total	5886

finances projects implemented by other UN organizations or by the recipient countries.

Among other important UN organizations is the World Health Organization (WHO) which assists with health programmes in the developing countries. The programmes include, *inter alia*, vaccination campaigns, setting up of systems for storage and distribution of medicine, and AIDS control.

The total development aid from the UN system was US$5.886 billion in 1992. The aid from the major UN aid agencies is listed in Table 22.5.

22.3.3 The European Union

EIB, EDF

Development aid from the EU is organized by the European Investment Bank (EIB) and the European Development Fund (EDF). The aid is provided according to the so-called Lomé conventions named after the capital in the West African country Togo. The Lomé conventions are framework agreements for co-operation between the EU and 70 participating developing countries in Africa, the Caribbean and the Pacific. Lomé convention IV was established in 1990 and covers a 10-year period. In 1992 the net disbursement by the EU amounted to US$4156 million.

STABEX

An important element in the Lomé conventions is the STABEX agreement (Stabilisation des recettes des exportations). This is an arrangement to harmonize the fluctuation of the revenue from exports of tropical farm produce to the European Union.

To assist the former Eastern Bloc countries the EU has established a special bank, the European Bank for Reconstruction and Development (EBRD) and the Cooperation Funds Progamme (CFP).

22.4 NATIONAL AID ORGANIZATIONS

Bilateral aid is transferred directly between the national aid agencies and the recipient countries.

Field offices

Most industrialized countries have established national aid organizations and have a field office in the developing countries who receive the most extensive assistance. The offices supervise activities funded by the donor countries and assist the authorities of the host countries in identifying and planning new projects. Normally the aid agencies' field offices form part of the respective embassies or high commissions. Some organizations are highly decentralized to the field offices while others are centralized.

Political criteria

Fundamentally, official development aid is based upon tax money and therefore is a domestic as well as an international issue. Not least because of the domestic political angle the weighting of areas of concentration must to a certain degree reflect domestic political criteria and also shifts in such criteria. This is obviously not without its problems. Recipient governments have their own criteria which may very well differ from the criteria established by the donor organization.

Donor's responsibility

Every developing country needs development of a very wide range of activities from 'grass-roots' projects in health, water and education to major investments in infrastructure and industry. It can be next to impossible for a poor country to reject proposed donor assistance for an activity although it lacks the recipient's priority. This places a high level of responsibility upon the donor to ensure that only locally prioritized and sustainable projects are supported.

Recipient's responsibility

On the other hand, no donor can or wants to work in all sectors and specializations are common for donors. This puts back the ultimate responsibility to the recipient for co-ordinating donor assistance to ensure that a balanced and sustainable development takes place. In practice this is almost impossible for the recipient and may lead to 'stop and go' developments in the various sectors or to complete negligence of some sectors for years on end. Accommodation of policy shifts in development work can also be very difficult because of the generally long time needed for preparation and implementation of projects.

22.5 AN EXAMPLE – DANIDA

Danida

An example of a national aid organization is the Danish International Development Assistance (Danida).

Organization

Danida forms an integral part of the 'South Group' of the Danish Ministry of Foreign Affairs. In 1993 the South Group employed a full-time staff of 360. Of these about 103 were on assignment at field offices in 16 co-operating countries. To supplement the permanent staff, Danish as well as local professionals, Danida co-operates with private companies, consultants, research institutes and private organizations in connection with all aspects of project preparation and implementation. The characteristics of Danida are explained in the following.

22.5.1 Objectives and principles

Preamble

The Danish International Development Act forms the basis for the official Danish development assistance. It states that the objective of Danish development assistance is to 'assist the developing countries in their social and economic development and in their political independence'.

Priorities

The preamble of the Act does not explicitly state that aid shall be directed at the

poorest segments of society in the poorest countries, but all the debates in the Danish Parliament during the last two decades concerning development assistance have emphasized the poverty orientation of Danish aid. Other basic requirements not expressed explicitly in the Act are sustainability and that women shall benefit directly from aid projects, or that assisted projects, at any rate, shall not have a negative impact on their situation.

Environment

All projects falling under Danish aid shall include due consideration to the environmental impact of the activities and an increasing number of projects have been initiated which aim directly at solving environmental problems.

Human rights

The human rights issue has, since a decision by the Danish Parliament in 1988, become an integral part of Danish development assistance co-operation. Respect for basic human rights is increasingly becoming a factor of importance for selection of countries with which Denmark is prepared to co-operate in the field of development assistance.

Human resource development

The training aspect in development work has been and remains a high development priority area for co-operation. Training is implemented as 'on the job training' and as formal training in training centres. Ultimately the development of human resources is a basic requirement for sustainability. The training aspect is discussed in Chapter 25.

Action plan

In 1988 the Danish Parliament adopted an action plan for development assistance. According to this plan loans to developing countries should be discontinued, and from 1989 all development assistance is provided in the form of grants. The aim shall be to use at least half of the bilateral assistance on procurement of goods and services in Denmark, but there is no administrative distinction between untied assistance and assistance tied to procurement in Denmark. The action plan also calls for decentralization of the administration to the local Danida offices (embassies) in existing and future main recipient countries.

Strategic plan

In 1994 a new strategic plan for Danish development assistance was approved by Parliament. The main elements in the plan are concentration on a limited number of 20 countries and selection of a few sectors in each country where support is to be provided in the shape of sector programme assistance.

Country selection

The main recipient countries in 1994 were Bangladesh, Benin, Bhutan, Bolivia, Burkina Faso, Egypt, Eritrea, Ghana, India, Kenya, Mozambique, Nepal, Nicaragua, Niger, Tanzania, Thailand, Uganda, Vietnam, Zambia and Zimbabwe.

Total aid

For 1993 the official Danish net total assistance was US$1.36 billion corresponding to 1.02% of the GNI which is more than the internationally agreed target for development aid from a donor country.

Traditionally, Danish aid has been distributed nearly evenly between bilateral and multi-lateral assistance. However, this policy may change, for example, in order to enhance Danish involvement in aid activities utilizing Danish aid funds and in order to provide funds for the former Eastern Bloc countries.

22.5.2 Multi-lateral assistance

Danish multi-lateral assistance in 1993 amounted to US$604 million or 44% of the total aid. 6% was used on information and research, etc. Major recipients of Danish multi-lateral assistance were UNDP (US$88 million), IDA ($68 million), EDF ($40 million) and WFP ($34 million).

Table 22.6 The six countries that received most bilateral aid from Denmark in 1993 (ref. 4).

Country	*% of bilateral assistance*
Tanzania	12.5
Uganda	6.5
India	4.8
Mozambique	4.7
Bangladesh	4.3
Zimbabwe	4.3
Total	37.1

22.5.3 Bilateral assistance

In 1993 Danish bilateral assistance amounted to US$680 million or 50% of the total aid. 62% of the bilateral assistance was transferred to the main recipient countries. The six countries that received most assistance are listed in Table 22.6.

Projects

The main part of the bilateral assistance (US$490 million or 72%) was used on project assistance, i.e. activities aimed at solving well-defined problems. The projects covered a wide field within the social infrastructure, economic infrastructure and production. The size of the projects ranged from mini-projects of a few thousand US dollars to big projects of several million dollars.

Programmes

In line with other donor agencies Danida is getting increasingly involved in programme assistance. Programme assistance may include commodity import support and other balance of payments support measures in connection with structural adjustment programmes. In 1993 the Danish programme assistance amounted to US$64 million.

NGOs

A growing part of the Danish development aid is channelled through private Non-Governmental Organizations (NGOs). This has been a deliberate government policy in order to mobilize the Danish resource base and widen the total scope of activities. In 1993 the Danish NGOs received a total of US$168 million from Danida's budget. Part of this assistance was financed using multi-lateral funds. The NGOs, such as the Red Cross, Church Aid, Volunteer Service, etc., are mainly involved in social and humanitarian work and not in direct investment projects such as infrastructure, including roads.

22.6 THE TRANSPORT SECTOR

Importance

The transport sector continues to play an important role in aid co-operation with the developing countries as a prerequisite for economic and social development. Realization of benefits from investments in the transport sector, however, nearly always require heavy and even larger investments in other sectors.

World Bank

The World Bank Group has always been strongly engaged in the transport sector. In 1993, US$3.8 billion or 16% of the World Bank's and IDA's commitments were for transport sector activities (ref. 1).

Regional banks

In 1993 the AfDB group approved loans and grants totalling US$370 million for the transport sector corresponding to 15% of the total volume (ref. 5). The AsDB in 1992 provided US$1.2 billion to the transport and communications sector corresponding to 24% of the total volume (ref. 6).

EU

The European Union under the current agreements (Lomé II to IV) has allocated a total of US$3.1 billion to transport and communications as of the end of 1992. As a percentage this sector has seen a gradual decline from 18.5% under Lomé II to 14.0% under Lomé IV (ref. 7).

Danida

In Danish development assistance the transport sector (road, rail, air and sea) has remained an important component. In 1993 this type of aid constituted 9% of the Danish bilateral transfers or US$61 million. Out of this, highway construction and road transport projects accounted for $40 million.

During recent years increased interest has been taken by donors in support for rural access roads and road maintenance. Also labour-intensive work methods instead of equipment and capital-intensive methods have been given priority. This has been motivated partly by the need for stimulation of economic activities in the rural areas, and partly in order to ensure sustainable road maintenance.

22.7 SUCCESS OR FAILURE?

Ownership

Whereas donor agencies and financial institutions have important roles to play as supporters and facilitators of development activities the recipient countries remain responsible for the activities, and project 'ownership' should be secured early in the process. Lack of commitment or 'ownership' has been a feature contributing to failures. The reasons behind lack of 'ownership' can be several, including a too-dominant role by the donor agency.

White elephants

Development projects sometimes hit the headlines in the media not least when an aid activity has resulted in a failure or has received the epitaph of being a 'white elephant'. As usual the story of failure and disaster sells better than those on activities that fulfil the objectives and quietly provide the results which recipient and donor together have agreed for the activity. This aspect of news dissemination tends sometimes to obscure the realities of the success or failure rates of development projects and generally donor agencies are not very good at advertising their activities.

Risks

Development aid must by nature be considered a risky activity and some of the risks are very difficult to assess at the time of project preparation and to include in the decision-making process. Added to this is the fact that the time from identification to completion of an activity may easily be around five years. This means that in an environment with fairly unstable political conditions an aid activity may still run into trouble even with the best possible preparations being undertaken before the decision to assist is taken.

Road and highway projects are also at risk of becoming 'white elephants'. One major risk factor is the inadequate maintenance and repair of roads and highways. In such a situation the original investment may become almost totally lost within some 10–15 years. Projects should carefully consider this issue, and if satisfactory solutions cannot be found a project idea may have to be given up.

Precautions

There are, however, several precautions which the two parties, the recipient and

the donor, can take in order to identify the risks and take measures to counteract the risks. Ultimately this may mean that the donor – or the recipient – declines participation. Every donor agency must do its utmost to ensure that all activities reach acceptable levels of success. It can also be argued that a 100% success rate would indicate that donors are too cautious in their selection of activities, and that some element of risk should generally be accepted.

Preparation

Most donor agencies and recipient governments do involve very considerable resources in connection with project preparation in order to achieve maximum benefits from their activities, and of particular importance are systematized and comprehensive preparatory studies and independent reviews and evaluations. The impact which the donor agency and the recipient can potentially have on a project's design and development is gradually reducing as the project preparations and subsequent implementation take place. The earlier in the process, the greater the potential impact and influence.

Evaluations

For completed projects or projects in advanced stages of implementation, evaluations are carried out, including evaluations of assistance to a particular sector. An evaluation is an investigation as systematic and objective as possible of preparation, implementation and outputs of a project or programme. The evaluations are undertaken by teams that are normally composed of persons from outside the aid agency and can present a critical analysis of all phases and aspects of the individual project or sector of support. Such evaluations which form the last step in the 'project cycle' are considered an important tool in bringing about a higher quality and lower failure rate to projects. The project cycle is described in the next chapter.

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23 The project cycle

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23.1 INTRODUCTION

Need for framework

Governments in developing countries and donor agencies deal with a large number of activities in various stages of preparation and implementation of donor funded projects which leads to the need for a comprehensive and systematic framework for the administration thereof. Due to the varying specializations of the different agencies and to the different frameworks and objectives for the various agencies each one, as well as governments in the developing countries, have their own systems and procedures. A large degree of uniformity has, however, developed over the years not least because of the efforts of the World Bank as a leading institution for development of theory and practices in development work.

As a result it is possible to categorize the various phases which an aid activity goes through, from when it is first conceived as an idea, to its completion and taking into use. This is done by application of a terminology which is generally understood and accepted by donor agencies, the co-operating partners in the developing countries, as well as private firms and institutions participating in the process. The various phases a project goes through from identification to completion is normally referred to as the 'project cycle'.

Naturally a clear line may not always be drawn between phases, meaning that some phases may contain elements of a preceding phase or the following one.

Co-operation

Project preparation and implementation is a process which is initiated by and under the overall responsibility of the government in the recipient country. It is a process which involves close co-operation first of all between the authorities of the recipient country and the donor agency, but also with private parties. This is especially so if it is a private sector activity, consultants, specialists, etc. A clear understanding of the elements involved in project work and the definition of the various phases thereof is a basic requirement for effective co-operation.

It could well be argued that it is the donors that have forced the formalities upon the recipient countries and that in other words the subject is a 'donor project

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cycle'. As the recipient countries develop their own administrations there is, however, growing recognition of the general need for a systematized approach to co-operation with donors. Not only is this likely to ensure a higher degree of success than less comprehensive and well thought through working methods, but it facilitates the working relationship between recipient and many different donor agencies with a more streamlined process. It also means that administrations in recipient countries, which may be under considerable local political pressure to get donor assistance for various projects, can utilize the formalized procedure to ensure proper advance investigations and possible dumping of unfeasible 'pet projects' motivated by local political aspirations.

Development plans

Most developing countries produce and up-date development plans covering five years or more and, depending upon the quality and capacity of the administration in the particular country, a considerable analytical effort may have gone into production of the plan. Normally, however, such plans contain a large number of projects which have been subject to analysis of widely varying scope and quality. Whereas most donor-funded projects in the public sector are selected from the official plans this does not necessarily mean that any of the steps in the project cycle can be eliminated.

Project cycle

The following description of the project cycle represents a basic description and as such will be common to many recipient countries and to most donor agencies, including the World Bank, the EU and the bilateral aid agencies of DAC member states.

Phases

The project cycle is divided into the following four main phases:

- identification;
- project preparation;
- project implementation;
- project completion.

Each main phase is subdivided into a number of subphases as illustrated in Figure 23.1.

Consultants and contractors

Throughout the project cycle it is normal that a number of consultants and contractors are involved, apart from the recipient government and the donor agency. The details of the procedures for selection of consulting engineers and contractors are described in Chapter 17. For major tasks the main rule is competitive bidding, but from time to time and generally where small tasks are concerned negotiated contracts are accepted.

In the following each phase in the project cycle will be presented and discussed.

23.2 IDENTIFICATION

23.2.1 Preliminary definition

Purpose

The purpose of project identification is to identify and describe one or more activities/projects which can assist in solving development problems or in satisfying recognized needs. In this phase of the cycle the description of the problems or needs, the underlying causes, constraints or risks are more important than the actual description of the proposed project.

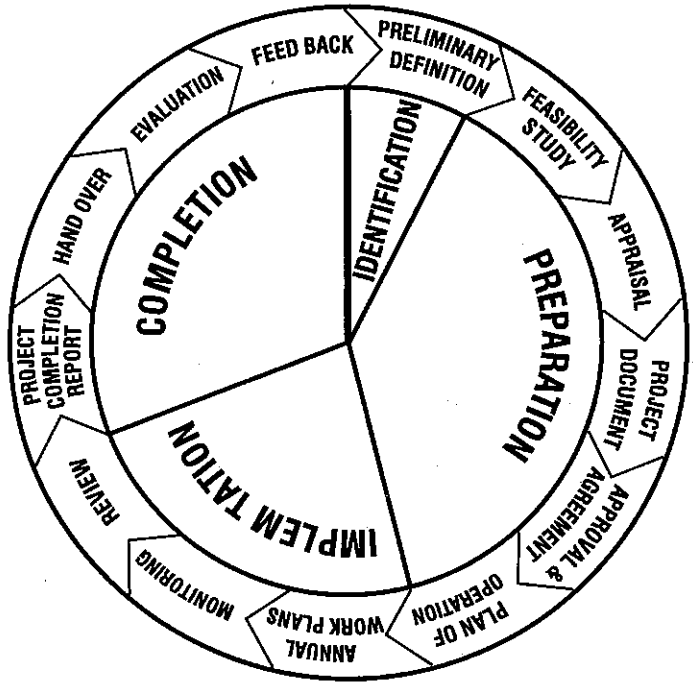


Figure 23.1 The project cycle.

Importance of quality

It is a common and natural experience that rejection of a project proposal becomes increasingly difficult the further into the project cycle the project idea has come.

Insufficient analysis and data gathering in this phase may lead to wrong decisions or considerable wastage of effort should the idea be dropped at a later stage. This does not, of course, mean that subsequent detailed assessment of the idea cannot lead to the idea being abandoned but underlines the crucial importance of high quality and sound judgement in this phase.

As part of the identification a close look must be taken at the needs aspect, and it must be ascertained that the proposal is in harmony with overall agreed plans and programmes of the recipient and with the donor's priorities with regard to sectoral emphasis, geographical areas, etc.

Donor co-ordination

It is important to ensure at identification that duplication of efforts is avoided and that donor activities in the relevant sectors are identified. It must be ascertained that the proposal is in accordance with agreed principles among donors for co-operation in that particular sector if such agreements exist.

Difficulties

Identifying a project that meets these requirements is not easy. Data on which to reach sound judgement may be lacking. The government and other donor agencies may not share the donor's views on development objectives or sector priorities. There may be difficult choices regarding the scope of the project (should it start with a pilot/experimental phase or with a larger but more risky investment?). Differences may quickly surface on the need for policy or institutional reforms to

achieve the project's objectives. A common obstacle is the conflict between a recipient government's often short-term, local political objective and the donor's more long-term and institution-building objectives.

Procedure

In practice, how are projects identified within this context? Both the government and the donor are involved, making the process complex, and this complexity is compounded by the differing capabilities of governments for handling economic planning and project generation. Judgement may be based in part on an economic analysis of a country prepared by the World Bank or others and is affected by the extent and quality of the country's data base and the donor's ability to prepare additional economic information. Often the World Bank as 'lead agency' may have taken responsibility for elaboration of sectoral plans with other donors arranging their contributions within this framework.

Private sponsors

Some projects are brought forward by private sponsors such as private industry seeking to develop new production facilities, etc. These projects have to meet the standards described above before being regarded as 'identified' from a donor's point of view and the financing arrangements need to be clarified.

Further elaboration

The outcome of the preliminary definition will often be that it is found necessary to further elaborate on the proposal before a firm understanding of the project idea can be established and passed on to the next phase of the cycle. Should this be the case then consultants are often engaged to assist recipient as well as donor with a project formulation exercise resulting in a more comprehensive description of the proposal. The responsibility for this activity often rests with the donor.

It must, however, be noted that the proposal may be dropped, based upon the original presentation or at any time during project formulation. If dropped during the identification phase this will happen after what is often referred to as a 'pre-appraisal'. If, however, the idea survives the identification stage it is passed on to the 'feasibility study phase'.

23.3 PROJECT PREPARATION

23.3.1 Feasibility study

The government of the recipient country is responsible for preparation not only of the project proposal but also of the supporting feasibility study.

Purpose

The feasibility study is an in-depth analysis which is undertaken in order to establish whether the proposed project will solve the problems which have motivated the identification, whether it has the proper scope and the relevant inputs, whether it is possible to implement and to sustain in the future after the donor has finished his involvement and whether alternative solutions should be considered.

The feasibility study analyses the proposal from many angles, such as technical, economic, financial, social, institutional, organizational and environmental. The economic analysis is described in Chapter 3. The feasibility study typically requires a multi-professional team as it addresses a wide-ranging complex of different and often interlinked issues. The study is normally undertaken by external consultants based on terms of reference agreed between the recipient and the donor. The consultants recommend under which conditions the project may be supported or not. These recommendations are not binding for the donor or the recipient.

Comparing alternatives

Feasibility studies must identify and prepare preliminary designs of technical and institutional alternatives and compare their respective costs and benefits. Most developing countries are characterized by abundant and low-cost labour and scarce capital. A donor will, therefore, look for technological solutions most appropriate to the country's resource endowment at its stage of development. Although one will find many examples of fairly technologically advanced development aid projects, consideration will have been given in each case to whether to support traditional wire borne telecommunications over cellular phone systems, modern dairies and dairy products over simple dairies and products, EDP-based land registration and modern mapping technology over traditional methods, and labour-intensive road construction over equipment-based techniques.

Draft project

Provided the feasibility study resulted in a positive recommendation a revised project proposal will be elaborated provided the proposal as such is still found viable. The revised proposal will be presented often in the shape of a 'draft project document'. If, however, the feasibility analysis has indicated various possible solutions the draft project document will be prepared only after a decision has been reached between recipient government and donor on which alternative should be pursued. The level of detail in the draft document permits economic, financial, technical and other analyses but is not sufficient for actual implementation. That level of detail is included in the 'project document' described later.

Time required

All this takes time, and donors are sometimes criticized for the length of time required to decide on a project. But for a donor and for the countries concerned each project represents a major investment with a long economic life, and the time spent in arriving at the best technical solution, in setting up the proper organization, in anticipating and dealing in advance with marketing and other problems, usually pays for itself several times over.

23.3.2 Appraisal

Purpose

Appraisal is the donor's assessment of the proposed project after it has passed through the feasibility phase. The appraisal is the last activity before going to the financial authorities to obtain funding and the purpose of the appraisal is:

- to make it possible for decision-makers to make rational decisions;
- to contribute to a well-prepared project design;
- to prepare a final project document.

Appraisal is a critical phase of the project cycle because it is the culmination of the preparatory work, provides a comprehensive review of all aspects of the project, and lays the foundation for implementing the project and evaluation when it is completed.

Appraisal is solely the donor's responsibility. It is conducted by the donor's own staff, often supplemented by individual consultants, who usually spend several weeks in the field. If the feasibility study has been done well, the appraisal can be relatively straightforward; if not, a subsequent mission or missions to the country may be necessary to complete the appraisal. The appraisal covers all major aspects of the project, including technical, institutional, economic, financial and environmental and establishes the donor position regarding the proposed project.

Technical appraisal

The technical appraisal is concerned with questions of physical scale, layout and location of facilities; the technology to be used, including types of equipment or processes; the appropriateness to local conditions of proposed technical standards; the approach to be followed for provision of services; the realism of the implementation schedule and the likelihood of achieving the expected levels of output. In a family planning project, the technical appraisal might be concerned with the number, design, and location of maternal and child health clinics, and the appropriateness of the delivery services in relation to the needs of the population being served; in highways, with the width and pavement strength of the roads in relation to the expected traffic, the trade-off between initial construction costs and recurrent costs for maintenance and between more or less labour-intensive methods of construction; in education, whether the proposed curriculum and the number and layout of classrooms, laboratories and other facilities are suited to the country's educational needs.

A critical part of technical appraisal is a review of the cost estimates, and the engineering or other data on which they are based, to determine whether they are accurate within an acceptable margin and whether allowances for physical contingencies and expected price increases during implementation are adequate. The technical appraisal also reviews proposed procurement arrangements to make sure that formal requirements are met. Procedures for obtaining engineering, architectural or other professional services are examined.

Technical appraisal is also concerned with estimating the costs of operating project facilities and services and with the availability of necessary raw materials or other inputs. The potential impact of the project on the human and physical environment is examined, to make sure that any adverse effects will be controlled or minimized to within acceptable levels.

Institutional appraisal

The institutional aspects of a project in the past have often been given insufficient attention which have led to problems during implementation and operation. Institutional appraisal must address a host of questions, such as whether the entity is properly organized and its management qualitatively and quantitatively adequate to do the job, whether local capabilities are being used effectively and can be made available to implement the project and operate the services after completion, and whether policy or institutional changes are required outside the entity to achieve project objectives.

Of all the aspects of a project, institution building is perhaps the most difficult to come to grips with. This is in part because its success depends so much on understanding the cultural environment. It is often difficult or politically and emotionally unacceptable for the recipient government to realize the extent of the shortcomings in the local systems.

Economic appraisal

The economic appraisal up-dates the feasibility study's economic analysis and passes a final judgement on the economic pros and cons of the proposed project.

The economic appraisal attempts to establish quantitative consequences of the project. The non-quantifiable benefits of a project must be described in sufficient detail to allow for an intelligent assessment of the pros and cons of alternative projects and solutions.

The non-quantifiable benefits may well be the decisive factor for choice between, for example, proposals with identical or similar quantifiable benefits.

A highway appraisal examines the relationship with competing modes of trans-

port such as railways. Policies throughout the transport sector are reviewed and changes recommended, for example, in regulatory practices and price setting that distort the 'modal split', i.e. allocation of traffic to different modes.

Some of the elements of project costs and benefits, such as pollution control, or better health or education, or manpower training, may defy quantification. In other projects, for example electric power or telecommunications, it may be necessary to use proxies, such as revenues, which do not fully measure the value of the service to the economy. In some cases it is possible to assess alternative solutions having the same benefits and select the least cost solution. In yet other cases, for example education, alternatives are likely to involve different benefits as well as different costs and a qualitative assessment must suffice.

Financial appraisal

The financial appraisal serves to ensure availability of funds for implementation, i.e. that a realistic financing plan exists. The financial appraisal, however, also addresses several other issues such as pricing and cost recovery.

The financial appraisal often highlights the needs to adjust the level and structure of prices charged for a service. Whether or not they are publicly owned (or parastatals), enterprises generally provide basic services which are politically sensitive. Sometimes the government wishes to subsidize such services to the consuming public as a matter of policy or perhaps simply as the line of least resistance. This in time may mean that the government is reluctant to approve the price increases necessary to ensure efficient use of the output of the enterprise and to meet its financial objectives. But adequate prices are a requirement to revenue-earning enterprises and the question of rate adjustments may be critical to the appraisal and subsequent implementation.

Financial appraisal is also concerned with recovering investment and operating costs in full or partly from project beneficiaries. This may lead to a situation where a balance has to be struck between the need for efficient utilization of scarce resources and the desirability of reaching a large number of potential beneficiaries.

Environmental assessment

Environmental impact assessment has become a standard requirement for most donor agencies' appraisals. The scope of assessment and degree of detail depends upon the expected environmental importance of the proposed project. The World Bank classifies projects in three categories (ref. 3):

- Complete environment analysis is required because the project may have a diverse and significant environmental impact.
- Limited environmental analysis is appropriate because the project may have an easily identifiable and manageable environmental impact, and
- Environmental analysis is not usually necessary because the projects are unlikely to have a significant environmental impact.

The environmental assessment should among other things determine whether the project:

- Affects areas with animal or plant life worthy of protection.
- Affects areas with significant historic and cultural remains or landscape elements of importance to the local population.
- Causes regressive or progressive soil erosion.
- Leads to high rates of use of scarce material resources.
- Changes the way of life of the local population in such a way that it leads to an increased pressure on the natural resource base.

- Leads to major conflicts with regard to existing land use and ownership of land.
- Increases noise and dust impact in the local area especially along unpaved roads.
- Causes illegal timber cutting and illegal land clearing.
- Causes illegal invasion by squatters and poachers of homelands belonging to indigenous peoples.

Appraisal report

The appraisal report must set forth its findings, conclusions and recommendations in clear, comprehensive and unequivocal terms as a basis for the ensuing decision-making process. On the basis of the appraisal often the project document is drawn up.

23.3.3 Project document

Following appraisal many but by no means all donor agencies prepare the project document. This document may deviate considerably from the original project proposal document with changes having been introduced as required by preceding analysis. The preparation of the project document is the donor's responsibility, but can only be finalized after having been subject to scrutiny and agreement by the recipient country's authorities.

The project document is of a high degree of detail and complete scope and may include legal, contractual and specialized sections. Its preparation generally requires a considerably larger work input than the draft project document.

Killer assumptions

In this context the subject of assumptions is critical and not least the so-called 'killer assumptions'. The document lists and describes the important assumptions upon which the recommendations for approval are based. Preparation of this part requires sound and mature judgement and not wishful thinking.

Risk assessment

Another important aspect of the document is presentation of the risk assessment based on the appraisal's results. Any project is subject to a number of risks, and the document must as clearly as possible spell these out and provide judgement as to the likely levels of probability of the risks having an effect on the project.

Implementation plan

An integral part of the project documentation is the plan of implementation. This is a comprehensive plan for the entire project with a degree of detail compatible with the complexity of the project. In large and very complicated projects it is sometimes decided to postpone the preparation of the plan of implementation to the very first activity of the implementation phase. An important element in the plan of implementation is the monitoring indicators which will make future monitoring of progress possible.

23.3.4 Approval and agreement

Presentation

Different donors have different systems for the procedures leading up to actual approval and provision of funds. It is common, however, for the presentation to the authorities to be based on the appraisal supported by specialized documents such as the project document and supplemented by customized presentation formats.

Approval

A presentation may result in an approval straight away or a conditional approval requiring certain changes, additional analysis, changes to local commitment, etc. More seldom, a presentation may lead to an outright rejection.

Government agreement

Provided that the funding for the project is approved the next step is to establish a government agreement for the project, as official development assistance is based

on government-to-government co-operation. Normally agreements are between a donor and the recipient's Ministry of Finance.

23.3.5 Plan of operation

In order to systematize and ensure an efficient execution the project document contains a 'plan of operation' which serves as a management tool. This plan contains detailed descriptions of activities and divisions of responsibilities between the various parties involved, work schedules, budgets, etc.

Appendix to government agreement

The plan of operation often forms part of the government agreement as an appendix. This has the advantage that the government agreement can be limited to the major features of the particular co-operation with the details laid down in the plan of operation. The plan of operation can be amended as mutually agreed between the donor and the implementing authority whereas changing the text of the agreement itself would normally be a lengthy procedure. By attaching the plan to the agreement it is, however, given high status to ensure adherence to it.

Time limit

Government agreements are normally given a time limit up to around five years and co-operation on the project beyond that period is thus subject to mutual agreement.

23.4 PROJECT IMPLEMENTATION

23.4.1 Annual work plans

In order to ensure the timely implementation of the project, detailed annual work plans are prepared based upon the plan of operation. In principle the implementing local authority is responsible for the preparation of the annual work plans, but in reality it will often be advisers or contractors involved with the implementation who draft the plans. The annual work plan provides the detailed implementation information, such as activity diagrams, budgets, etc. The preparation of annual work plans provides a framework making revisions to the original overall plan of operation as required in response to unforeseen difficulties.

Donor's supervision

It is general experience that once a project is under implementation the recipient's interest turns towards preparation of new projects which are being considered. This adds importance to the donor's supervisory role which is normally not a particularly glamorous one.

No matter how well a project has been identified and prepared its development benefits are realized only when it has been properly executed. All projects face implementation problems, some of which cannot be identified in advance. These problems may stem from difficulties inherent in the development process as well as from more specific causes such as changes in the economic and political situation, in project management or even in the weather. As a result, although the development objectives of a project generally remain constant, its implementation path often varies from what was envisaged.

Supervision takes place in a variety of ways. During negotiation, agreement will have been reached on the schedule of progress reports to be prepared by the implementing agency and which cover the execution of the project, its costs, the financial status of revenue-earning enterprises and information on the evolution of project benefits.

Progress reports
Site supervision

The progress reports and site visits form the basis for the donor's project supervision utilizing the monitoring indicators established at the time of appraisal.

Site supervision by the donor is often placed in the hands of consulting engineers or other consulting firms, and progress reports are reviewed at the resident mission and problems sorted out locally or at head office level as required.

23.4.2 Monitoring

Monitoring

Monitoring is the term used for the local implementing organization's current supervision of the project. The purpose is defined as the provision of the necessary information for project management, needed by the recipient and the donor in order to adjust activities, inputs and budgets to ensure achievement of project objectives.

23.4.3 Review

Periodic reviews

During implementation periodic reviews take place under the donor's responsibility, but often are undertaken as joint reviews, i.e. jointly with the recipient.

A review is a comprehensive assessment of the implementation as seen against the project document, the government agreement and the annual implementation plans. The purpose of the review is to solve accumulated problems and to set out clear guidelines for the project's development until the next review.

23.5 PROJECT COMPLETION

23.5.1 Project completion report

Completion report

Once a project is completed a project completion report is prepared under the responsibility of the donor. The report provides a systematic description and evaluation of the preparations, implementation and results of the completed project with the purpose of documenting the project cycle and extracting valuable experience. Normally the project completion report is prepared without field visits and is based on existing documents.

23.5.2 Hand-over

The completed project is handed over to the local authority responsible for the project, 'the end user'. The recipient thereafter assumes full responsibility for the facilities and activities which will continue as a result of the achieved outputs.

Depending upon the nature of the project the hand-over will include the physical hand-over of assets such as buildings, vehicles and machinery, transfer of know-how and rights, etc. The hand-over may take place in various ways such as from contractor to recipient government who in turn hands over to the 'end user'. Often the hand-over has important legal implications regarding rights to operate equipment, guarantee periods, etc.

23.5.3 Evaluation

'Ex post'

Most development assistance evaluations are carried out 'ex post', i.e. some time

after the project is completed. An evaluation is a systematic and objective assessment of preparations, implementation and results.

Purpose

The purpose of evaluations is to assess the appropriateness of the objectives, the rate of achievement of objectives, effectiveness and sustainability. The results of an evaluation must be of a high degree of trustworthiness and must be presented in a form that makes the findings useful to be incorporated in the decision processes of the donor as well as the recipient in connection with new projects. The evaluation report is the most important single feedback document in development work and is essential also to the other participants in the process, such as consultants, contractors, etc.

Consultants

Most donor agencies have established autonomous evaluation units and utilize independent consultants to carry out the evaluations. They are undertaken as a combination of desk and field studies which may involve gathering of primary field data. The main emphasis is on gathering and dissemination of experiences which are useful in relation to the specific activity, but also in a wider context for preparation and decision on future activities.

In World Bank evaluations it is customary for the recipient to be requested to prepare a separate section with an evaluation of the Bank's performance.

23.5.4 Feedback

Lessons learnt

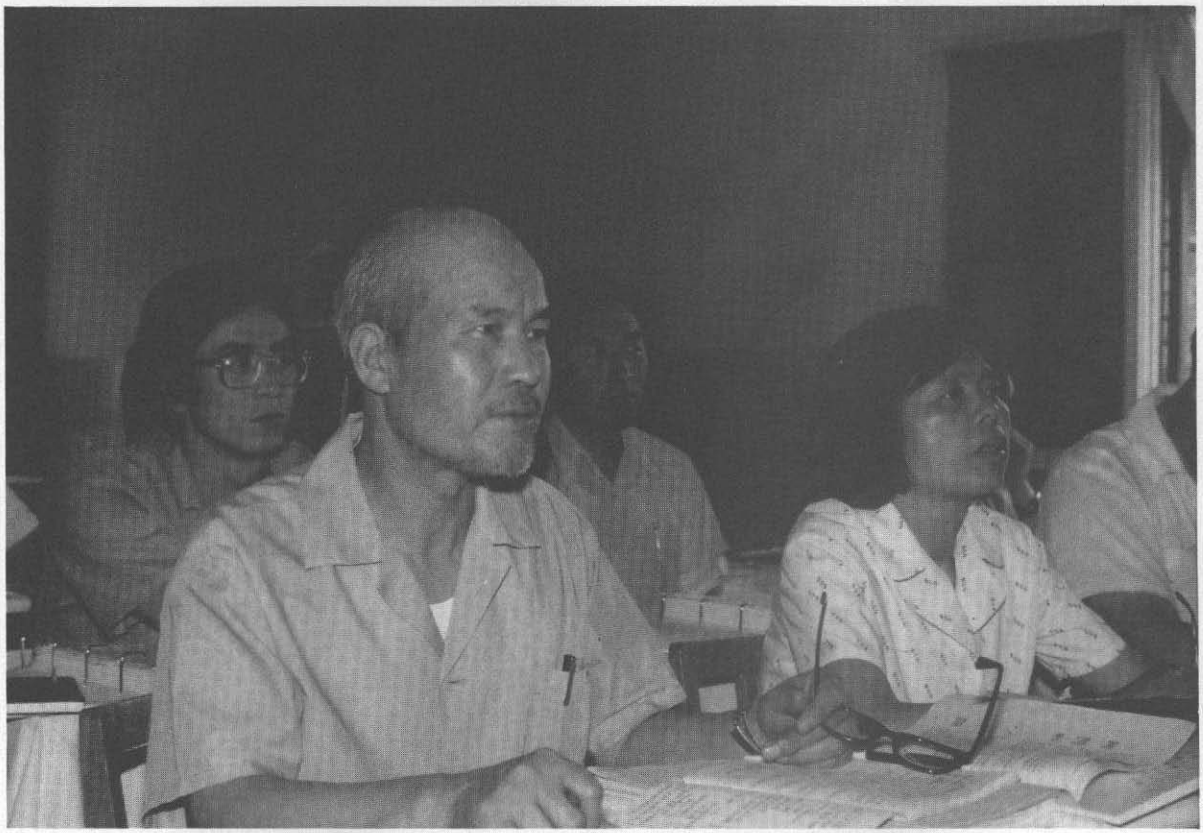
Through the evaluation the lessons learnt are fed back to the parties involved and built into the design, preparation and implementation of future projects. In this way the project cycle is completed.

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PART 10

Institutional Issues



Attentive trainees in China. (Photo by Bent Thagesen)

Institutional strengthening

24

Paul S. Taylor, Cowiconsult

24.1 INTRODUCTION

Definition

Institutional aspects of road development cover a range of issues concerning both (a) the general policy environment, that is, overall government objectives and strategies for national development, in which the organization operates (Section 24.2), and (b) the road agency itself (Section 24.3). Issues pertaining to the policy environment include the relationship between the road agency and other affected organizations, the role of government, and the role of the private sector. Of concern in the latter case are matters relating to the internal structure of the agency, its functions, responsibilities and operating procedures.

Over the last decade, there has been a growing recognition of the importance of such issues to the performance of the subsector. Consequently, much emphasis has been placed, particularly by donor agencies, on determining appropriate institutional arrangements and on strengthening institutional capacity. Rural access roads, which have their own special problems, have been the subject of particular interest from donor agencies, and have therefore merited separate treatment (Section 24.4). Mobilizing local involvement in planning, paying for and undertaking roads operations, has also been an emerging theme over recent years. This topic is dealt with in Section 24.5.

Context

Studies of road operations in developing countries have consistently highlighted deficient institutional arrangements as a major reason for poor performance in the subsector. Insufficiency of resources alone cannot explain poor performance.

In a recent review of 127 rural access roads projects carried out in Sub-Saharan Africa between 1964 and 1989 (ref. 1), institutional problems were identified as by far the most common cause of poor project implementation, affecting 35% of the projects. Common problems were the lack of organizational focus leading to administrative fragmentation and duplication of responsibilities; insufficient technical and managerial expertise; little accountability for financial and physical performance; poor staff motivation and lack of career opportunities.

An assessment of deterioration of the main road network in 85 developing countries (ref. 2) again identified the effect of poor institutional development

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on operations. The report showed that inadequacies stem in part from the monopolistic structure and functions of the traditional road agency. Such failures have often been linked to the road agency having too many responsibilities – for planning, controlling and executing both construction and maintenance. It typically devotes too many staff, funds and facilities to execution, to the detriment of planning, control and evaluation.

Maintenance

The major outcome of the failings of roads institutions has been that they have not adequately maintained existing networks, maintenance being a management-intensive activity that requires an effective institutional set-up. As a consequence, many nations, particularly in Sub-Saharan Africa, are embarking on expensive schemes of road rehabilitation, which could have been avoided if workable systems of maintenance had been established. Meeting the need for effective maintenance arrangements, rather than implementing ambitious rehabilitation projects, is the most important institutional challenge being faced by developing nations. This priority will be reflected throughout much of this chapter.

Solution

It is clear that there are a number of common institutional problems affecting road operations in developing countries. Studies have shown, however, that there are no standard solutions to such problems; rather, institutional development has to proceed on the basis of a local, pragmatic approach, taking into account the specific, country context. Nevertheless, there is a set of common issues which need to be addressed in establishing and/or strengthening the institutional framework for road development. These are presented in the following sections.

24.2 POLICY ENVIRONMENT

Private sector

An emerging theme of institutional reform is an increasing emphasis on the involvement of the private sector, not only in undertaking construction, rehabilitation and maintenance, but also in ownership, financing, and management (ref. 3). At one end of the spectrum of engagement of the private sector are toll roads owned and operated by businessmen and entrepreneurs, and at the other, the introduction of private sector practices and disciplines into road administration. Most recently, the World Bank in particular has been reviewing the opportunities for promoting independent road authorities. These are intended to be run along private sector lines, with private sector conditions of service for staff, and full accountability for efficient road operations.

Road financing

The new approach has also entailed a fundamental review of the way in which governments finance roads operations.

It is obvious that unless funds are made consistently available to meet the costs of the entirety of roads operations, maintenance of the road network is not possible, with the result that the network will decay. Thus, either road users must pay the full costs of their use of roads, or subsidies must be made available from elsewhere in governments' budgets. The latter is not a realistic option for many Third World governments, who are often struggling with crippling fiscal deficits.

Nevertheless, in the past governments have been unwilling to adopt an explicit policy of cost recovery for the use of roads. This has been due to a mixture of (a)

political considerations resulting from the unwillingness of road users to pay the higher prices that are often required by a policy of full cost recovery (increases in fuel taxes not infrequently cause political unrest and have even caused governments to fall); (b) the view that roads are a public good and therefore must be accessible to all, not just those who are able to pay; and (c) diversion of revenues raised from road users to meet requirements, particularly for expenditure that cannot be met from user charges, for example, health, education or defence, elsewhere in governments' budgets.

Sustainability of the network therefore generally implies that there must be cost recovery from road users. Fuel taxes are the obvious source of funds and these should be set at levels sufficient to ensure an adequate flow of funds into the subsector.

Road fund

For cost recovery policies to be effective, there should be a seamless and transparent transfer of revenues from road users and allocation for expenditure in the subsector. If this link is broken, it is likely that tariff setting and expenditure levels will be distorted. Traditional earmarking is not enough, as governments have, when faced with revenue shortfalls, often diverted funds earmarked for roads, or frozen them, regardless of road expenditure requirements. A road fund is often proposed as an enhanced earmarking mechanism which facilitates automatic transfers of road revenues from road users to road expenditures, the fund providing the budget for road authorities.

However, governments are often reluctant to embrace such fiscal allocation principles, because it can limit their room for manoeuvre within national budgets. Furthermore, it is feared that the principle will be applied to other areas of taxation and expenditure where an equivalence between revenue collected and money expended is less desirable. It is of note, however, that there is tendency throughout the Western world to make such linkages due to wide dissatisfaction with systems of generic taxation and government mediation in expenditure.

Impact

Evidence tends to suggest that where revenues are linked to expenditure and there is also private sector involvement, road standards are significantly better and costs tend to be lower. For instance, studies of the condition of toll roads in Mexico and Korea demonstrate that their condition is consistently better than on other roads. Comparable studies of toll roads and bridges in the USA have produced similar conclusions. Studies in France indicate the costs of designing and constructing private toll roads are 10–15% cheaper than public toll roads.

Current division of responsibility

The above description deals with most recent thinking on the introduction of market concepts into roads administration. Currently, a more typical division of responsibility for road works entails construction and rehabilitation being undertaken by private contractors overseen by a public sector road agency, although a number of countries maintain large direct labour or force account units for these purposes. To a large extent, particularly in relation to large projects, the use of contractors has implied utilization of foreign firms as local contracting industries have generally not been well developed.

Force account

Most types of maintenance are undertaken by the road agency itself using direct labour, otherwise known as force account units, which consist of workers hired directly by government without the involvement of contractors.

Increasingly, however, the responsibility for recruiting, mobilizing and sustaining

a large work force has become a burden on the public agency. Force account work has often been of poor quality due mainly to weak management, poor equipment maintenance and lack of properly trained staff, and carried out at higher costs.

Private contractors

Greater reliance on the private sector for construction, rehabilitation and maintenance has thus been advocated, with a number of advantages being apparent. For instance, evidence shows that contractors can often provide the required labour and equipment at lower costs and with less delay than government organizations. In one example, with fully comparable costs under similar conditions (Ponta Grossa, Brazil), maintenance by force account cost 60% more than contract maintenance. Small contractors can often operate more easily and cheaply in remote areas than a central road organization. Furthermore, experience suggests that private contractors use and maintain machinery more effectively.

It has, therefore, been concluded by many agencies that the use of private contractors offers better prospects for developing an efficient and sustainable capacity for carrying out road operations. Unfortunately, where road operations are undertaken by government ministries or departments, there are often serious constraints on the use of the private sector imposed by government policies and procurement regulations. Fostering the domestic contracting industry, on the one hand, and reducing force account operations, on the other, will therefore require strong government commitment, involving changes in policy and administration.

Government attitudes

Many governments have in the past encouraged the hiring of large (not to say bloated) numbers of force account workers as a conscious measure to reduce unemployment, or less officially, as a mode of patronage. Reversals of such practices are not lightly contemplated, especially in the context of World Bank and IMF sponsored structural adjustment strategies, which often create generally increased unemployment and consequential social and political unrest.

Example

Ghana provides an example of a country in which government has introduced both a policy change in the road sector and the practical support needed to implement the policy. Thus, the Government has swung towards promoting the use of labour-based contractors to undertake road rehabilitation as one of the means by which it can combat unemployment created by the shake-out of the economy resulting from a structural adjustment programme. Support is given through training of labour-based contractors, secured provision of initial contracts, and making available loan finance for the purchase of equipment. As a result labour-based contracting is operating efficiently and effectively, bringing kudos and government praise to the sponsoring road agency, thus reinforcing the agency's commitment to promoting the use of private contractors.

Supporting measures

In the realm of practical government action to support such policies in the road sector, capacity-building for local contractors would require the provision of training programmes to cover cost estimating, preparation of bids, site management, methods of execution of roadworks, contractual obligations, etc. Government may also need to consider measures involving positive discrimination towards local contractors, for example, allowing a suitable preference in bid evaluation, by easing procurement systems.

Small contractor development will also have certain institutional implications for the road authority itself. For example, it increases the need for supervisory management and inspection of works; it requires reliable financial systems to

ensure prompt payment for work. These latter two points will be discussed in more detail in later sections.

Alternative technologies

An allied issue which has implications for institutional arrangements is the choice of work methods for construction, rehabilitation and maintenance. In the previously mentioned review of rural access roads projects in Sub-Saharan Africa, the accepted technology for construction and rehabilitation, and sometimes also for maintenance, has been equipment-based. In only 16% of rural road construction projects and 19% of road maintenance projects were labour-based methods used.

However, there have been some notable successes in labour-based techniques – in Kenya, Ghana and Malawi. Coupled with the endemic problems associated with equipment use, the adoption of labour-based technology has been seen by other developing countries as a viable alternative to the latter, incurring lower costs and benefiting rural people more.

Policy implications

Experience has shown that, where labour-based operations are indicated, government will need to make a clear policy commitment for change. This will call for special institutional arrangements, comprehensive planning and effective managerial and administrative systems and procedures. Above all, flexibility in managing site workers is required. Sometimes well-intentioned government worker protection legislation will preclude taskwork and piecework labour, prescribe inflexible working periods, and require high minimum wages for formal work. These factors may combine to make labour-based operations less feasible. Labour law reform may therefore be required if labour-based operations are to be successful.

Decentralization

If there is a political commitment in a country to the principle of decentralization of decision-making responsibilities, this may need to be balanced against the other objectives of roads administration such as operational efficiency, accountability of line managers, and sustainability of operational arrangements. Decentralization, particularly where local administrations can formally insert overtly political considerations into decision-making, can conflict with these objectives. Consideration has therefore to be given by national governments to what responsibilities are most appropriate to decentralize and to what level in the administrative hierarchy.

National planning

The planning system should constitute the means by which government allocates and manages its resources at the national and sectoral levels; and it should provide the framework within which implementing agencies can plan and budget at a more detailed level. But evidence from a review of thirty years of planning in developing countries suggest that this rarely happens (ref. 4).

Planning problems

Overall resource planning and allocation has often been absent; public investment plans and the national budget are prepared in an unco-ordinated manner; there has been little incentive to undertake full and close scrutiny of the recurrent budget; and the formulation of development plans is often unrelated to financial and implementation capacity and to subsequent maintenance requirements.

Improvements in the management of resources for road operations is, therefore, dependent upon strengthening planning and budgeting capacity at the national level. Some improvements have been taking place. In particular, the discredited 'blueprint' approach to planning has now been largely abandoned. However, the

alternative 'learning process' approach, in which consultation, flexibility, selectivity, monitoring and evaluation are key elements, is taking time to become institutionalized. But, if new systems are to become effective, capacity must be built up at the regional and local levels as well. This will lead to a less centralized, more responsive, 'bottom-up' approach, which is likely to result in better plans at the end of the day.

However, it must be borne in mind that there is a considerable technical input into road planning. Assuming local administrations are involved in the planning process (and most evidence suggests they should be at least for rural access roads), this technical input can best be communicated by professional staff of the roads agency. This creates a strong imperative towards internal decentralization within such an agency.

Salaries

The desirability of strong involvement by professional staff in roads administration and planning at the local level is mitigated against by certain other factors. In many developing countries, public service salaries are extremely poor for professionally qualified personnel. This is often due to government policies on depressing salary differentials. The result has been an inability to retain experienced professional staff within public service roads organizations. As a result there are often insufficient qualified and experienced staff to provide local level support. Governments will be unable to improve the quality of roads administration until this fundamental problem is addressed.

Centralized bureaucracy

Increased effectiveness of roads administration is likely to require augmentation of the capacity of field administration. But, good staff are unlikely to be attracted to and retained in field positions if career opportunities, higher salaries and other incentives are concentrated at headquarters, as tends to be the norm. These anomalies need to be addressed by governments.

Other incentives

If salary inequalities cannot be reduced because of rigidities within the public service system, careful attention will need to be given to other incentives for field staff. For example, access to housing, in situations where normal housing supply mechanisms have generally broken down, can be a powerful substitute for salary increases.

Training

A major feature of many roads institutions is the critical lack of qualified and experienced staff. Budgets for training are generally inadequate. No scheme of institutional upgrading can be successful without a major staff training component.

Staff motivation

However, lack of training is often a fig-leaf used by managers to explain poor staff morale and motivation, which is likely to be the result of antiquated rules on promotion or seniority, or of advancement based on patronage and nepotism. Government, while supporting training, must also be prepared to tackle these other problems as well, otherwise training benefits may be lost.

Technical assistance

Most donor supported programmes are accompanied by the provision of technical experts in advisory roles whose job is to help upgrade the long-term performance of the roads institution. The World Bank comments that the result of this form of assistance is that 'providers of technical services have little exposure to risk, few incentives for improving performance, and only indirect responsibility (if any) for measurable output' (ref. 2). Concerned agencies will have to give much more consideration in future on how to extract the maximum value from technical assistance.

24.3 ROAD AGENCY INSTITUTIONAL ARRANGEMENTS

24.3.1 Trunk roads

Organizational structures

The above discussion has outlined some of the wider issues which need to be taken into account when the specific institutional arrangements and performance for road operations are considered. The following sections consider some typical organizational structures, highlighting their inherent problems and identifying ways of overcoming them through strengthening institutional capacity.

Trunk roads

The institutional arrangements which characterize highway administrations generally reflect the fact that trunk roads are of national importance and that matters relating to them, such as priority-setting, road design and maintenance standards, depend to a large extent on technical input. The scope to undertake planning at a local level, or to involve local communities in construction or maintenance, is limited. Typically, therefore, highway administration is the responsibility of a centralized road agency which undertakes all the functions of planning, budgeting and execution.

Such an administrative structure has, in the opinion of the review, *Road Deterioration in Developing Countries* (ref. 2), contributed to the inadequacies of road maintenance experienced in many of the 85 countries studied. The review argues that such an organization has both too many and conflicting objectives: it operates in a monopolistic way and lacks accountability. Internal decentralization, together with the functional separation of responsibilities, are examined as possible solutions.

Separation of functions

The review argues, however, that there is little correlation between success in trunk roads maintenance and the degree of internal decentralization of trunk roads responsibilities. At the trunk roads level of the road hierarchy, it is accepted that a relatively centralized system may be required. This conclusion may not apply to all countries, especially to those which are large and where telecommunications are poor. In conventional government road organizations, in accordance with the review, it is suggested that the most obvious need is for the separation of planning and supervision from the execution of works.

In institutional terms, this implies strengthening the capacity of the central road agency to plan, supervise and monitor works, which will be executed by another, independent agency. Experience has shown that it is desirable that this independent agency should be the private sector. It should become increasingly involved in the execution of both construction and maintenance works.

Road authorities

More radical solutions are now being mooted since the above-cited review was undertaken. These entail an even more 'arm's length' relationship between government and roads administrations. Independent or quasi-independent road authorities, run on private sector lines, or structured so as to provide opportunities for private sector equity participation or management, could be given contracts to administer all or part of the network. These contracts would be performance related, and should performance criteria not be met, there would be opportunities for the government to apply penalties or seek new contractors (ref. 5). The advantage of this arrangement is that road authorities' inputs (money) and performance targets would be made explicit, and the road authority should be given every possibility to exploit the flexibility provided by freedom from govern-

ment procurement systems and conditions of service for staff to maximize its achievements.

The linking of roads authorities to dedicated road funds is intended to lead to a situation whereby consumers (or customers) who purchase their road use in turn expect value for money, and have an accountable institution to lobby. This is expected to create a market discipline for the road authority.

Management information systems

A prerequisite of the central road agency's enhanced planning and supervisory role, or of a road authority performance contract system, is the establishment of an internal management information system (MIS). Through the planning process, resource allocations should be linked with action plans setting physical and financial targets. Through reporting and monitoring procedures established within the management information system, progress towards achieving these targets can be measured and corrective action taken by management where necessary. Data generated by the system can be used to assist in the analysis and establishment of unit costs, which can, in turn, be fed back into the planning and budgetary process and improved manpower management in search of increased productivity, the latter being an essential element of any performance contract set-up.

The building up of a database, an area for possible international funding, should over time enable the undertaking of ex-post evaluations to measure the longer-term effects and impacts of road projects.

Chapter 21 deals with maintenance management systems (MMS). It is emphasized that an MMS is a subset of an overall MIS. An MIS is concerned with overall organizational performance, while an MMS is almost entirely concerned with engineering and network considerations.

24.3.2 Rural access roads

Greater institutional complexity than in trunk road operations is involved in rural access road programmes. For this reason, more detailed discussion of the latter is presented, with some examples drawn from studies of Sub-Saharan African countries being used for illustrative purposes.

The administrative arrangements for rural access road operations are often characterized by a plethora of agencies: central road authorities, regional administrations, local government, special project units set up in ministries or within parastatals, amongst which there is, on the one hand, generally little co-ordination and, on the other, duplication and overlap. Experience shows that this leads to a number of significant problems. A typical distribution of responsibilities for rural access roads is accordingly described in some detail below.

Example

In one African country regional administrations used to have all the planning, budgeting and execution responsibilities for regional roads. A regional engineer, based in each regional capital, programmed and supervised all works on regional roads. The regional engineer was on secondment from the central road agency but was responsible to the regional administration in all matters.

District councils were responsible for the planning, budgeting and execution of district roads and unclassified access roads. A district engineer, who was normally employed under local government conditions of service, had technical responsibility for operations and was directly accountable to a district executive director. District

engineers were not normally qualified professional engineers but were usually technicians.

A recent organization and management study of operations highlighted the problems and implications resulting from the above distribution of responsibilities, which, in turn, directly led to very poor road conditions, high vehicle operating costs, and adverse economic consequences for agricultural marketing, pricing and profitability.

The main problems were as follows:

- lack of institutional focus, that is, an excessive number of agencies involved in administration of the sector;
- lack of technical and managerial expertise;
- duplication of effort and inefficient use of resources;
- inconsistent application of technical standards;
- poor motivation and lack of career opportunities;
- poor financial performance.

Lack of focus

Clearly, the lack of an institutional focus implicit in such organizational arrangements as described above hinders rural access roads development due to the fragmentation of policy and implementation. The fragmented nature of the roads operations also makes institutional reform much more difficult.

Lack of expertise

It is common in many developing countries that there is a severe shortage of qualified and experienced road engineers. Institutional arrangements have compounded this problem since they have led to the inefficient utilization of trained personnel. Thus, much of the district level roadworks, which need supervision by suitably experienced engineers, is undertaken by technicians who lack appropriate training and skills. Those engineers who could be available to provide advice and direction are, however, responsible to an entirely separate administrative structure.

Duplication

In the above example, there was no co-ordination between the organizations involved in road operations on matters such as planning, programming of work or use of equipment. For instance, the central road agency maintained two entirely separate workshops at the regional level, one for trunk roads and the other for regional roads.

Inconsistent standards

The fragmented administrative structure meant that there was no single responsible agency which could enforce the application of appropriate technical standards of rehabilitation and maintenance throughout the country. Consequently, standards that were technically inappropriate to deal with a specific situation were often adopted.

Lack of career path

The administrative arrangements offered no channel of support to professional staff from the central technical agency nor a clear career path. Staff morale was adversely affected.

Financial performance

Disbursement of funds through a variety of different agencies using diverse accounting procedures, with varying degrees of expertise, led to lack of accountability and poor financial performance on road projects.

Alternative approaches

In the light of the kind of problems outlined above, an important issue which has to be considered in the effort to develop institutional capacity is whether to try to strengthen existing roads institutions to carry out rural access roads operations or to create new institutions for this purpose. Both approaches have their drawbacks.

Special units

A typical example of the latter approach has been the creation of a special project unit within an agricultural ministry or a parastatal, established to deal with road projects, often with expatriate technical support. Experience has shown that, although often offering excellent technical performance, such units have 'generally failed to show any lasting effect in terms of developing institutional capacity to implement or maintain improved rural roads' (ref. 6). Indeed, they are often disbanded upon completion of works with no future maintenance arrangements being made since the issue of classification and management of access roads after project completion is rarely clarified at the project preparation stage.

Existing agencies

Modifying existing road agencies to take responsibility for rural access road operations also has disadvantages. Such agencies often carry the 'baggage' of an inward looking and protective institutional culture and will be resistant to change. Furthermore, the track record of these agencies or ministries may not obviously lead to the conclusion to broaden their involvement in road operations.

World Bank experience has shown that, on balance, it is preferable to modify existing institutions to meet new needs rather than to create new, separate institutions. Indeed, the institutional arrangements adopted in a number of the most successful rural road operations in developing countries reinforce this conclusion.

Kenya

Thus, in Kenya, the Rural Access Road Programme (RARP) has been operated through a Special Programme Branch of the Road Ministry. 'It succeeded because it was placed within a ministerial structure that already had a well established record for administration of roads' (ref. 1).

Ghana

In Ghana, a Department of Feeder Roads was established under the auspices of the Ministry of Roads and Highways to be responsible for rural road operations.

Tanzania

On a similar basis, an enhancement of the organizational capacity of the existing Ministry of Communications and Works through the creation of a Rural Roads Division was the key feature of institutional change for rural access roads operations in Tanzania.

24.4 RURAL ROADS DIVISION

Institutional arrangements for rural access roads have been the subject of particular scrutiny over recent years as it has been in this area where road deterioration has been particularly pronounced. Attention is therefore turned to the details of how rural roads operations could be carried out.

Reform of host agency

Incorporating rural access roads responsibilities into an existing national roads institution, as advocated above, often has to be accompanied by changes to the management set-up of the host agency. The changes required in institutional outlook often run counter to the centralist and bureaucratic mentality of national roads administrations, whose outlook was formed by the paternalistic approach of colonial and immediately post-independence administrations. Implementing them, therefore, has often proved difficult.

The new role will normally stress the role of the agency in managing and planning operations, decentralizing management responsibilities wherever possible, relinquishing as many force account operations as possible, and developing its expertise in supervising the operations of private contractors.

Internal structure

Whatever arrangements are determined for the implementation of a country's rural access roads programme, issues concerning the internal structure of the selected executing agency, the division of responsibilities, and systems and procedures, all of which will affect institutional performance, must also be considered. These matters are amplified below.

Clearly, the centralized system associated with trunk road operations is far less appropriate in relation to rural access roads programmes. The review of rural access road programmes in Sub-Saharan Africa (ref. 6) concluded that the most desirable set-up was that of a small centralized rural roads unit placed within an established roads agency. This arrangement has achieved better results than those obtained by just adding rural access road responsibilities to the current duties of existing units within the roads agency. The rural access roads unit would oversee a regional organization which would in turn be responsible for local planning and operations. Issues such as decentralization and delegation of responsibilities and the role of local participation are thus of particular relevance in this context.

Experience has shown that this institutional arrangement provides the necessary professional and technical focus for rural access road operations and the channel for allocating funds earmarked for the subsector; it facilitates the adoption of innovative methods of planning, construction and maintenance; and can help to foster an esprit de corps among its professional staff. 'Such an organization enjoys the legitimacy and the channels of communication of the larger agency, while at the same time becoming a focal point for introducing institutional change' (ref. 6).

Tanzania

By way of illustration, in Tanzania, it was concluded that a Rural Roads Division should be set up at headquarters to co-ordinate activity, but that day-to-day operational responsibility for rural access roads should be decentralized to the regional level as much as possible. This implied a clear specification of respective roles and responsibilities between the newly established Rural Roads Division at headquarters level in the Ministry of Communication and Works and the regional offices of the ministry.

Headquarters

The Rural Roads Division in Tanzania was made responsible for overall strategic planning for the rural access roads subsector, acting as an efficient channel for the flow of allocated funds, providing guidance to the regional level on planning and budgeting for rural access roads, monitoring and quality control of implementation performance, standard-setting and training.

Regional offices

An enhanced implementation role was envisaged for the regional offices with much more involvement in the planning and programming of rural access roads and greater management responsibility for operations.

Procedures

Revised procedures and systems for planning, budgeting, disbursement of funds, and monitoring will need to be developed to support the arrangements described above.

Planning

In the majority of the developing countries, the development of rural access roads has been marked by the absence of any comprehensive planning. Decisions on priorities and on the allocation of resources between construction, rehabilitation and maintenance have normally been made on an ad hoc basis. Indeed, maintenance planning has more often been neglected or, at the least, considered separately from construction and rehabilitation. There has been an over-concentration on economic returns as the primary criterion of evaluation and selection. Clearly, there is a need for the establishment of a proper planning framework and system.

In the terms of the recent World Bank review of rural access roads (ref. 2), the development of a country strategy for rural access roads together with a 'multi-tiered planning and programming systems based on locally acceptable criteria allowing participation of local communities' is required.

The development of planning methodologies and evaluation criteria represents a task for headquarters to undertake but in co-operation with the regional office using its knowledge of local conditions.

The application of such criteria would form part of the annual planning and budgetary process co-ordinated by the regional office. Based on road priorities determined at the regional level, preferably (as discussed later) incorporating community preferences, action plans would be drawn up. The regional office would then determine its required level of resources through the preparation of its annual budget proposals.

Budgeting

The establishment of a rural access roads strategy would also facilitate the annual budgetary process. Based on priorities contained within the strategy and implementation progress, annual resource targets could be set and disaggregated down to the appropriate level for the preparation of detailed budgets and action plans.

Typically, co-ordination and review of annual budgets and action plans, and integration into a rural access roads strategy, would form a major part of the strategic planning function of the rural roads division at headquarters.

Disbursement of funds

Closer linkages between the processes of strategic planning, and detailed action and budgetary planning, together with input from all relevant constituencies, may discourage the subsequent, arbitrary cut-backs to, and reallocation of, road funds which commonly occurs currently.

It has been argued that one of the advantages of establishing a separate rural roads branch is that it can act as a channel for funds allocated for the rural access roads subsector. But mechanisms must also be in place which will ensure that there is then a smooth flow of these funds throughout the financial year from headquarters to the executing agency of the rural access roads programmes. One of the procedures to facilitate this would be the preparation of a work programme setting out an implementation schedule, together with a cash flow statement, by the executing agency for submission to the head office. These would form the basis of the release of funds; normally this would be undertaken quarterly with the provision of monitoring reports on financial and physical progress being essential to subsequent disbursement.

The regular and timely availability of funds at the field level of operations is also crucial, particularly if the use of labour-based methods and of the domestic contracting industry becomes more widespread in future. Prompt payment is, of course, also dependent upon the existence of efficient payment systems. Systems would need to be devised to avoid the over-concentration on procedures designed to minimize financial malpractice, which appears to be the norm currently, and to ensure prompt and correct payments.

Monitoring

One of the main bases of improved planning, budgeting, disbursement of funds and management of resources is access to information on financial and physical implementation progress. This implies the establishment of a reporting and monitoring system, which should be designed to satisfy the needs of its users at the different levels of the administrative structure using information generated initially from the field level of operations.

In a decentralized regional structure, such as has been outlined above, a monitoring system will constitute an important management tool in the day-to-day operations of rural access roads programmes. It will enable physical and financial implementation performance to be tracked, measured and evaluated. Where progress is not going according to schedule, a local manager is then in a position to take prompt corrective action, to reprogramme work activities and to redeploy resources accordingly.

The monitoring system will not only enable the executing agency to have better control of its operations but will also allow headquarters to play a more effective monitoring and quality control role. In addition, monitoring reports will be the means by which other agencies, such as local administrations, are kept informed of, and respond to, the implementation performance on road programmes in their areas.

Cost accounting

Cost accounting arrangements should form part of the monitoring system since they are concerned particularly with budgetary control. Cost accounting establishes budgets, standard costs and actual costs of operations and processes. Cost accounting responsibilities should be allocated to appropriate cost centres, that is, the lowest distinct level of activity with regard to inputs and outputs. In the case of road operations, this should be the unit responsible for implementation, for example, a regional office or area office. The manager of this unit should be accountable for ensuring that satisfactory output is achieved with the inputs provided.

24.5 LOCAL PARTICIPATION

Funding

With continuing lack of finance and trained manpower for rural access roads development from the centre, the mobilization of local resources has become an essential element for future operations. Funding of maintenance in particular is normally regarded as a local responsibility. In many local government areas this is not practicable to achieve at a level which will ensure the maintenance of a viable network. The reasons for this are legion. Examples are local government mismanagement, corruption, or overmanning leading to unwillingness to pay local taxes; or impoverishment due to poor returns from agricultural activity. Donors, in their turn, have traditionally been unwilling to provide financial support for maintenance operations. It is in this context that grass-roots participation in roads operations, especially maintenance, becomes critical.

Planning

Participation can occur in rural access road programmes in the planning process and in project implementation through involvement in construction, rehabilitation and maintenance activities. Experience has shown that, when local constituencies have been involved in the planning stage, there has been more success in mobilizing local resources for physical operations, particularly those involving a labour contribution.

Ghana

In Ghana, the Department of Feeder Roads has been responsible for establishing a hierarchical structure of committees for the purposes of community participation in planning, as well as for involvement in voluntary road maintenance. A District Road Committee (DRC) is at the head of this hierarchy; it is chaired by the district secretary and includes representatives of the major interest groups in the district. It is responsible for co-ordinating district level input in the planning and

budgeting process for access roads, discussing priorities and reconciling conflicting demands. These views are then taken into account by the regional engineer when formulating regional budget proposals. The committee also has a very specific planning role, playing a major part in selecting roads to be the subject of recurrent maintenance.

Kenya

Participation in the road selection process has also been determined as a contributory factor to the relatively successful planning of Kenya's Rural Access Roads Programme (RARP). Under RARP, proposals for rural road improvements were initially identified by District Road Committees (DRC) according to criteria set by the Rural Roads Branch of the Ministry of Transport and Communications. These proposals were then screened by a district engineer and subjected to preliminary technical and economic analysis, based upon data provided by the DRCs. Successful roads were then prioritized and programmed within the constraints imposed by the level of resources available to each district.

Such approaches reflect the move away from a concentration on purely economic criteria as the main tool for evaluation and selection of rural road projects towards a process which involves multiple factors including population, area, production and social, economic and cultural matters. The development of simple, well-defined road selection criteria should facilitate local participation in the planning process.

Routine maintenance

Attempts at achieving local participation in the maintenance of rural access roads has mixed results. This is seen to be due to a number of factors including government attitude towards community input, the lack of appropriate channels for participation, and the absence of institutional arrangements to facilitate involvement. Although such constraints exist, it is now accepted that the mobilization of local resources is a critical factor towards overcoming the problem of road maintenance – particularly routine maintenance – in developing countries.

Ghana

The approach adopted in Ghana towards routine road maintenance seems to address the issues raised above. With the shortage of locally sourced funds for maintenance, the utilization of voluntary community labour has been encouraged, building upon the strong tradition of community works in the country.

As referred to above, the Department of Feeder Roads has devised a hierarchy of committees, which link to the political district structure, in order to facilitate involvement in both selection of roads and voluntary road maintenance works. The first tier, the DRC, has already been described. The second-tier committee, the Central Road Committee, is in charge of formulating arrangements for the allocation of routine maintenance responsibilities for sections of the network to specific communities. Village Road Maintenance Committees then have the responsibility for ensuring that the physical maintenance of a specified length of the network is achieved.

The organizational aspects of implementation comprise a DFR district foreman working with the village committee, providing technical assistance, training, management support, quality control and tools to the villagers. The method of operation depends upon community preferences – in some instances, it involves a gang of 8–10 people working on a full-time basis, in others, a lengthsman system is used. The district foreman reports up to an area engineer, who is in charge of routine maintenance within the DFR's regional structure.

Two further observations can be made about the above example. First, the

community routine maintenance schemes are only being implemented in districts where the committee structure and other organizational supports have been introduced. Secondly, the development of community maintenance capacity should be integrated with the planning of rehabilitation programmes for an area. In other words, it becomes increasingly important that future rehabilitation projects take place only where the prospects of labour-based routine maintenance operating efficiently are reasonable. In Ghana this is often achieved by getting a formal commitment to the undertaking of routine maintenance by the local community before rehabilitation of a road commences. If no commitment is given the project may not proceed.

Routine maintenance

In Ghana an interesting approach to routine maintenance of rural access roads has been adopted. The Department of Feeder Roads maintains a fleet of graders. These are deployed on roads selected by District Assemblies. However, local communities must pay for all fuel and provide accommodation and food for drivers if the roads selected are to be graded. The district secretary will oversee operations to ensure efficient utilization of fuel and quality of output. Villagers are prepared to provide financial and material support as they can see a demonstrable relationship between their expenditure and the results.

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25 Training of staff

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25.1 PURPOSE OF TRAINING

Definition

The discussion on training in this chapter is concerned principally with training of staff of a road organization who are already employed or newly recruited. But the chapter will also be of interest to staff in consulting and contracting companies, and in donor agencies. Thus, in this context, training is different from education, and from pre-employment vocational and technical training.

Aim

The aim of training is to improve job performance by extending knowledge, inculcating skills and modifying attitudes, so that individuals can work in the most economical, efficient and satisfying way. At the corporate and the individual level, training may be considered as:

- a sequence of human-related activities which enables the road organization to perform delegated responsibilities effectively and efficiently;
- enabling a staff member, or a group of staff members, to conduct delegated functions and tasks within the road organization competently, and consistent with the corporate performance requirement.

The focus on the corporate performance will, to a large extent, ensure that an organizational culture is developed, enhancing motivation, co-operation, communication and appropriate management systems.

Needs

There are many reasons why employees need to be trained (ref. 1), including:

- shortage of labour;
- high turnover of staff;
- change in organizational practices;
- expansion or diversification of activities;
- need to improve the quality of work;
- need to raise the calibre of staff;
- prevention of deterioration in performance;

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- establishment of organizational bases in new locations;
- new employees.

Recruitment

The road organization will normally have a preference for recruitment of technical personnel from nationally recognized universities, colleges and training institutions in order to obtain the most relevant qualifications and skills possible. There may, however, be a number of constraints. The availability of qualified personnel may be insufficient. The mobility of professionals and technicians may be low. The employment incentives may be inadequate to attract the most qualified personnel. The taught qualifications and skills may not be compatible with the performance requirements of the road organization. There will also be a need to train administrative and clerical staff. In addition, many employees will be at the subprofessional or manual worker level, who will have few, if any, formal qualifications: training will be needed for these if their work is to be carried out efficiently and effectively.

Own training institutions

In many countries, the curricula at the universities, colleges, etc., have not been revised for two or three decades, and the education and training have become obsolete. To correct for this situation, the road organization in some countries has established its own education and training institutions which function at the national level.

Private sector

In countries with a well-developed private sector, there may be a high staff turnover in public agencies as the private sector may offer more attractive employment opportunities. This increases the need for training in the roads organization since staff must be recruited to replace those who leave, and new staff may not possess all the required qualifications and skills. With the increased focus on privatization in recent years, several governments in countries where the private sector is emerging have also organized training programmes for private contractors. The need for training will increase further if the educational background for certain disciplines is limited and the skill base for recruitment is small.

Scope

Training is a way of improving performance by changing the way that work is done. It is an indispensable requirement for improving resource allocation. Training transfers technology for:

- management techniques;
- technical matters;
- work methods and practices;
- procurement procedures.

Training is not discretionary; it is an economic necessity. It should also be a continuing function and part of an organization's strategic development.

Strategic role

Training, in particular, is a vital component in addressing the 'road deterioration problem' discussed in Chapter 21. Only by effecting changes in the way that maintenance organizations carry out their responsibilities – from top managers to field and shop personnel – can progress be made. The strategic role of training is to effect improvements in the way that highway organizations' work is actually performed, how equipment is operated and maintained, and how materials are used in the field. Ultimate solutions are long term in nature, but there are things that can and should be done in the short term. There is a general need for training programmes that are properly targeted at those activities which will have a direct

impact and give an immediate return. For many road organizations, these will include increasing productivity and service levels, and reducing foreign exchange needs. Training needs analysis is discussed in Section 25.4.

25.2 INSTITUTIONAL ISSUES

Achievement of training

Training is often prescribed as a panacea for transferring technology, but the World Bank have stated (ref. 2) that training has a poor record of achieving institutional development: undue emphasis has been placed on inputs (such as the number of people trained) rather than impacts (such as the effects of training). Training projects need to be formulated with considerable care if sustainable results are to be obtained. In particular, training should always be designed to meet clear objectives, with achievement that is measurable. Training needs to be developed in the context of the organization's wider aims and objectives.

Institutional appraisal

There is a high degree of correlation between training issues and institutional issues. Experience has shown that an institutional appraisal, covering organizational, managerial and technical issues, is the most effective method of identifying clearly training objectives and institutional constraints. In particular, an institutional appraisal will help to develop a common understanding of what issues the training should address in order that its objectives will best suit those of the organization. Thus, the institutional objectives and arrangements are deciding factors of imperative importance for the design of training interventions. No training should be planned and conducted in the absence of an institutional appraisal. The training needs assessment and the institutional appraisal should, ideally, be undertaken simultaneously.

Such appraisals identify the client's objectives and standards in the area under consideration. It is then possible to work backwards from these to identify any human resource constraints that might be impeding the attainment of these objectives. The optimal situation is to achieve a situation where there is a balance between the institutional objectives and the institutional capacity. This means that the technical and financial resources should correspond with the responsibilities and tasks delegated to the road organization.

Human resource development

A further reason that the record of training in developing countries has been disappointing is that, too often, training has been seen in isolation from the broader subject of human resource development. Insufficient attention is paid to establishing a framework of priorities to ensure that training is cost-effective and results-orientated. More attention needs to be given to manpower analysis before training is planned, and to manpower management after training has been completed. Insufficient analysis of manpower resources and needs carries the risk that the wrong type of training may be given, or that people are trained for jobs that are of low priority or do not exist. Poor manpower management means that trained staff are unable to apply what they have learned effectively, and their training is wasted.

The availability of trained engineers, technicians and managers varies from country to country. But even where well-trained staff exist, institutional performance has tended not to improve significantly because these personnel have not been effectively developed, utilized and retained. Whereas the earlier focus was on

training, it has become apparent that the human resource problem is more a question of utilization, motivation, development and retention (ref. 3).

Causal factors

There are a variety of factors which may cause ineffective use of manpower, including personnel policies resulting from the application of Civil Service rules, conditions of employment and pay within the public sector agencies, the lack of accountability and incentives, the level of the organization's efficiency and structural complexity. Such factors can affect the general ability to attract, retain, train and motivate technical and managerial staff. The result is often a demoralized work force, with apathy pervading everyone from senior management, through to equipment operators and labourers. Against such a background, human resource performance, and indeed training, is always likely to be ineffective.

Changing situations

There are, currently, many institutional changes taking place in road organizations in developing countries. These include:

- decentralization;
- increased private sector participation;
- adaptation of labour-intensive technologies;
- more emphasis on road maintenance.

Decentralization

Decentralization will result in the lower level of the road organization assuming greater responsibility for planning, implementation and maintenance. This is in contrast to the road organization's previous position in which all technical and financial management were undertaken at the central level. Decentralization will imply a restructuring and recomposition of staff. Some of the requirements for skills and qualifications cannot be met by training alone. It may be necessary to employ new categories of professional and technical personnel to cope with the changed delegated assignments.

Privatization

Increased private sector participation in construction, rehabilitation and maintenance will imply a reduction of the operational staff in the road organizations and, thus, fewer personnel. On the other hand, the staff remaining will require increased management skills in dealing with consultants and contractors for the preparation of tender documents, preparation of contracts, supervision, and financial management. The private consultants and contractors, on their part, will need to be provided with the necessary skills for bidding, work planning and contract management.

Labour-intensive technologies

The adoption of labour-intensive technologies also demands different skills and qualifications. Labour-intensive technologies are management intensive, whether the labour is employed directly by the road administration or by a private contractor. As with privatization, staff in roads organizations will, therefore, need to develop more in the way of leadership and management skills, whereas fewer technical staff and operators will be required.

Maintenance

A reallocation of more resources to maintenance will also require increased management inputs, as well as organizational changes. A systematic approach to maintaining roads according to identified needs will require training in the use of management systems, cf. Chapter 21.

National policy

The above issues should preferably be dealt with at the national level, and be incorporated in policies and strategies for the transport sector. Even where a national policy is absent, the process of transition and change may very well already be under way. So, even without clearly defined policies, it is important to

be aware of the general trends in the structural adjustment in government services at large, and in the transport sector in particular. The common element in the training needs that will be required as a result of the changes that are taking place is that much more emphasis will need to be given to management training than was the case in the past when technical training tended to dominate.

No standard solution

Chapter 24 concluded that there are no standard solutions for institutional development, and that the solutions must be based on a pragmatic approach in accordance with the prevailing environment. The same conclusion applies to training interventions as they go hand in hand with institutional development.

25.3 TRAINING TYPES

Definitions

Training can be delivered internally or externally, or as a combination of both. Internal training is provided on location within the institutional framework of the road organization. Internal training is provided in the road organization's own training centre or as in-service training. External training is provided in national or foreign training institutions. Training may be funded locally, or may be project-related and carried out as part of a project that is funded internationally.

Internal training

Internal training provides better opportunities than external training for adapting the training interventions to local needs, and it interferes very little with the trainees' social life. Internal training can take place without disrupting the service of the road organization significantly, and it can be conducted in the right context. Internal training may comprise on-the-job training, classroom training and counterpart training.

On-the-job training

On-the-job training emphasizes practical skills and routines, e.g. operating machines and instruments, which are learnt by observing and doing the actual work under the supervision of a skilled and experienced operator.

TPU

A special form of on-the-job training, often used in road construction and maintenance, is the training production unit (TPU). A TPU is attached to a training centre and is delegated the responsibility for construction and maintenance of a certain part of the road network. The TPU is composed of engineers, foremen, equipment operators and mechanics, who undertake the assigned construction and maintenance work. The TPU has all the necessary equipment and tools, and access to workshop facilities. TPUs provide a well-focused form of on-the-job training.

Classroom training

Classroom training deals with the theoretical aspects associated with particular tasks. On-the-job and classroom training are often related to manual and technical job categories, but classroom work also plays a key role in management training.

Counterpart training

Frequently, foreign staff assigned to a development project will have local counterparts. The counterparts are expected to take over when the foreign staff leave. Counterpart training is an informal way of transferring know-how during the daily co-operation when executing the work. It is normally related to middle and higher management job categories.

Client organizations

Unfortunately, the success of counterpart training has been very poor. On the part of government, counterparts are often assigned late, or only temporarily, to consultants. Often they are not assigned at all. Sometimes the counterparts may not be suitable for the type of training given or for the post that they may later hold. In some cases, foreign staff simply act as 'gap fillers'. There are often good

reasons for the lack of counterparts as this extract from a consultant's report illustrates (ref. 4):

> 'The availability of staff for retraining and upgrading was limited by operational needs and turnover. There was a higher percentage turnover of engineers, technicians and skilled operatives. In most cases, experience has been replaced by inexperience and there remained a continuing and substantial deficiency of establishment. The shortage of technical staff was exacerbated by a policy of leaving posts open, i.e. unfilled, for staff who left (the client's organisation) to work overseas.'

Consultant organizations

Counterpart problems are by no means restricted to problems in the client organization, as the following report extract shows (ref. 4):

> 'A central difficulty, never resolved, was the staffing of technical assistance consultants. The initiation of the project was delayed for over a year. The consultant staff originally proposed for the project made other commitments. Low consultant salaries plus the worsening economic conditions (locally) made it difficult to attract qualified experts. Since the elements of the programme were inter-related, delays in filling positions and poor performance in those positions made it certain that the balance of the programme would be affected.'

Sometimes, although consultant staff may be technically competent, they lack the personal skills required to transfer technology in a counterpart situation. However, in many cases, consultants are under substantial commercial pressures to complete the project, and counterpart training is viewed both by them and the client as superfluous to this end. Many project time-scales are simply totally inadequate to meet physical project objectives and to train staff.

Requirements

To be effective, counterparts should be assigned throughout the consultancy period, must be motivated, and must have the knowledge and capability to develop to the required level. There must, therefore, be incentives for development, and counterparts should be able to see that they will eventually be engaged in the position for which they are being trained, and will be able to apply the expertise that they have acquired. Best results are likely if, as quickly as possible, the roles of consultant and counterpart are exchanged, with the consultant then filling a largely advisory position.

External training

External training may be provided in a national training institute, in a regional training centre operated by several countries or in an overseas training centre. The advantages of external training are that more training resources can be made available, and that exchange of ideas from different contexts can be made if people from other domestic or foreign road organizations participate in the training.

National/ regional training

A national training institute may either be an engineering college, a management institute, or a training centre geared specifically to the road sector's training needs. An example of a regional training centre is the Eastern and Southern Africa Management Institute (ESAMI) in Tanzania, which is operated by Tanzania, Kenya, Uganda, Zimbabwe, Djibouti and Comoros. ESAMI offers courses in international contracts and planning of road maintenance. Participants come from member and other countries.

Overseas training

Overseas training is frequently offered by donor countries. This could either be post-graduate courses of one or two years duration at universities, short courses

offered by the national roads administrations, or short fellowships tailored to meet specific skills requirements. Overseas training is sometimes preferred by the bilateral donors to promote cultural exchange between the countries. Overseas training is also attractive to many trainees as it broadens their experience and develops maturity. Overseas training is aimed mostly at medium to higher skill levels, including engineers, technicians, surveyors, accountants, economists, etc. This type of training is relatively more expensive than local training, but overseas training may be the only solution if the necessary training staff and training facilities cannot be provided in the native country.

Example

An example of overseas training is the courses offered by the Danish Road Directorate. Three-month courses are arranged for Third World professionals in road and bridge management. The Danish International Development Assistance (Danida) has separate financial provisions for courses of up to three month's duration. Fellowships are normally linked to projects for which Danida is providing technical and financial assistance and, thus, fall under the category of project-related training.

Project-related training

In the past, the need for training was only identified when the performance of the road organization was declining. Training projects were then implemented to improve the performance. However, experience has shown that training is necessary as an integrated part of most development projects in order for the projects to succeed. For example, provision of new workshop facilities for repair of road equipment must include training of the mechanics who are to use the new facilities; introduction of a new pavement management system must be followed by systematic training of all the personnel who will be operating the new system. Training is particularly important for projects which also include an institutional strengthening component. However, experience from many projects suggests that training needs are often underestimated by a considerable amount.

Projected-related training tends to be very specific in its objectives as it is usually being delivered to attain well-defined performance requirements. Project-related training is likely to be of a short-term nature.

Twinning

The twinning of institutions in developing countries with similar, but more mature, organizations in other parts of the world has proved to be an effective way to transfer knowledge, train staff, and build management capabilities (ref. 5). Professional relationships between operating entities offer advantages of being complementary and are flexible over time. The entity supplying technical assistance uses its own resources to offer services to its twin in the functional areas in which they both work.

Twinning arrangements in the highways field have been carried out between the US Bureau of Public Roads and authorities in Ethiopia and Turkey; a further example is between Staffordshire County Council in the United Kingdom and the City of Accra in Ghana. Where twinning arrangements have been successful, there has always been an initial commitment and consensus on goals of the technical assistance. It is important to recognize that twinning alone may not be enough, and that other types of technical assistance may also be needed. Entities are not necessarily compatible simply because they are in the same business. Both client and supplier may need help in setting up a twinning agreement and working together to carry it out. A tripartite arrangement between a host organization and an offshore institution in conjunction with a consultant seems to offer the best

combination of skills and resources for solving institutional problems and assisting with development in this way.

Continued training

There are particular needs for continued training for operation and maintenance in the period following the completion of a project. In the past, less emphasis has been put on operation and maintenance by the national governments as well as the donor community. This has been largely due to a focus on development projects as opposed to operational aspects of the road system. The national development budgets have frequently targeted new road projects, and the budgets allocated to the subsequent operation and maintenance have been inadequate. Road organizations are, today, confronted with the task of establishing a more appropriate balance between development and maintenance.

Training not associated with physical projects may contain a higher element of general education than project-related training. This type of training has generally a longer duration.

Blend of training

The relevance of internal and external training and the appropriate type of training will vary according to training objectives and target groups. Often the most relevant training will be a blend of different types of training. The right blend is one of several considerations to be made in the planning process.

Training trainers

It is often appropriate to train counterpart staff who are capable of undertaking future training activities themselves, without outside assistance. This requires the provision of both consultants and counterparts who are not only motivated, but who also have appropriate interpersonal skills.

25.4 TRAINING NEEDS ANALYSIS

25.4.1 Determining needs

Identification

The demand for training may be identified by the road organization or by a donor in co-operation with the road agency. The scope for training, whether it is project-related or not, may vary from training at the local level to training concerned with functions of the entire road network. Thus, there is considerable variation in complexity of the types of training interventions that may be considered.

As noted earlier, a training needs analysis, carried out as part of an institutional appraisal, is the most effective method of identifying training objectives and institutional constraints. The analysis should comprise:

- institutional issues;
- training environment;
- current performance levels;
- required performance levels;
- training needs.

Current performance

The current performance level has to be established for each category of target groups or position in the road organization, and these must be evaluated in relation to corporate objectives and performance requirements. The current performance level is evaluated by interviewing representative sections of the personnel and by reviewing job descriptions. Figure 25.1 is an example of a manpower and skills

MANPOWER AND SKILLS INVENTORY FORM

Name of Employee:
Personal No.:
Age:
Job title:
Salary grade:
Length of service:
Years in present post:

Normal duty:
Exceptional duty:
Work supervised by:
Staff under supervision:
Promotion potential: supervisor's opinion:
management's opinion:

Education level:
Certificate obtained:
Post-school education:
Certificate obtained:
Skill training:
English: spoken: good/adequate/none
reading: good/adequate/none
written: good/adequate/none

Skills: in which proficient:
others:

Figure 25.1 Manpower and skills inventory form (source: Cowiconsult, Consulting Engineers).

inventory form. Figure 25.2 shows a job description for a district maintenance engineer in Nigeria.

It is also important to evaluate the performance level that can be expected from future employees. This may demand an analysis of existing schools and training institutes. Figure 25.3 is an example of a questionnaire that has been used for evaluating technical schools in Zimbabwe.

Training needs

The detailed training needs for each category of personnel are then assessed by comparing current and required performance levels.

25.4.2 Training priorities

The training needs analysis is likely to identify more training than can be afforded, and decisions must be made about which to undertake now and which to defer. The aim should be to set training priorities on the basis of cost–benefit principles: the choice of training to be undertaken should be that which gives the greatest return on investment. All potential training should be evaluated in this way. Examples of the use of a cost–benefit approach to prioritizing training have been given by Jorgensen (ref. 6). These have been used as the basis of the following example, illustrated in Table 25.1.

Example

Consider the following situation concerned with the grading of a network of unpaved roads: (i) each grading of a road is carried out using six passes of a

TECHNICAL OFFICER (Periodic Maintenance)	
Place of work	Office and Field
Work supervised by	District Maintenance Engineer
Staff under supervision	Technical Assistant and Specialist Overseers
Normal Duty	Instruction of Road Overseers Supervision of work and Reporting Planning Works Programmes Control of Production Units Ordering materials Allocating manpower Allocating machines and Vehicles
Exceptional Duty	Emergency Works
Details	Resurfacing Surface dressing Gravel production Operation of Quarry Bridge & Culvert repair, etc. Maintenance Road Furniture Minor Works

Figure 25.2 Job description for a district maintenance engineer in Nigeria (source: Kampsax, Consulting Engineers).

grader; (ii) but the graders are only available for use in operational condition for 20% of the normal working hours; (iii) as a result, 75 graders are needed for maintaining the network of roads. A training programme for grader operators, designed to teach them an improved work method which can produce the same result using only four passes of the grader, results in a saving of US$1.25 million. Alternatively, if improved equipment maintenance can increase the availability rate to 50%, the number of graders needed can be reduced from 75 to 30, with an estimated saving of US$2.25 million. If both operators and mechanics are trained, the total saving will be US$2.75 million.

Although this example is dramatic in its impact, this approach to evaluating the benefits of training can and should be applied to all activities in order both to justify and to prioritize training needs.

25.4.3 Complementary requirements

Not all technical capacity problems can be solved by training alone. When corporate performance standards are established, complementary interventions to training are likely to be required to reach desired standards. As stated earlier, there is a high degree of correlation between training issues and institutional issues. But it may also be necessary to introduce new technologies and new management systems. The training should not be seen in isolation. If complementary requirements are not met, the intended change may not occur.

1. SCHOOL Name: Location:
2. EDUCATION Content of Curricula and Syllabuses: Quality: Teaching Methods: Examinations:
3. STUDENTS Entry qualifications: Guidance of students: Staff/student relationship:
4. STAFF Staffing Policies: Selection and Qualifications: Staff Development and Training:
5. PHYSICAL RESOURCES Teaching Accommodation: Provision of Equipment:
6. RELATIONS with construction and maintenance units in the field:
7. ORGANIZATION Organization and Structure: Objectives and Plans:
8. EFFICIENCY Admission rate/drop-out rate: Time to produce a graduate: Staff/student rate: Accommodation Utilization: Costs:
9. NATIONAL SYSTEM Central Policies: Control of Courses: Control of Staff: Sources of Funds:

Figure 25.3 School questionnaire (source: Cowiconsult, Consulting Engineers).

Table 25.1 Alternative grader fleet sizes under four scenarios of work method and availability rate.

	Scenarios			
Work method	*6 passes*	*4 passes*	*6 passes*	*4 passes*
Availability rates	*20%*	*20%*	*50%*	*50%*
Required fleet size	75	50	30	20
Fleet investment* (US$ millions)	3.75	2.50	1.50	1.00

Note: *Assumes US$50 000 per grader.

25.5 PLANNING

Following the definition of objectives, institutional appraisal and priority needs analysis, a plan of implementation should be prepared. This plan should specify:

- criteria for selection of trainees;
- type of training to be used;
- training programme.

However, all planning must recognize the environment in which the training will be carried out.

25.5.1 Training environment

Definition

A number of factors influence the impact which can be achieved by training interventions. These factors may be termed the training environment and are concerned with supply, mobility, and trainability of manpower.

Supply of manpower

The supply and mobility of manpower are strongly influenced by the prevailing education system in the country. An elitist education system which concentrates resources on higher education for the few tends to have a lack of middle-level technicians and skilled workers - in contrast to an education system which aims at a more appropriate balance of primary, secondary and higher education. The geographical dispersion also affects manpower availability.

Mobility

More highly-educated personnel usually have a preference to remain close to the educational centres where they received their education. As higher education institutions are normally concentrated in the capital, or in a few large cities, it may prove difficult to attract higher-skilled personnel to more remote areas. The mobility can be increased by providing incentives. Public sector wages are generally low and the macro-economic restructuring of the public sector in many developing countries means that there is little scope for substantial salary increases. Incentives other than high wages include: career prospects, a challenging work environment, housing, etc. Provision of these may increase the mobility to the more remote areas.

Trainability

Trainability is concerned with the extent to which training can make an impact on staff's attitudes, qualifications and skills. The trainability relates to quality of education and work experience of the individual staff member.

Influence

The above factors will generally be outside the control of the roads organization or of a project concerned with performance improvement. But some of them may be influenced by the project activities. For example, a project-related training intervention may have an attachment to a training or an education institution dealing with road engineering and may, in consequence of the interaction, assist in modifying educational curricula.

It is important to have an understanding of the training environment as it will have to be incorporated in the design of the training intervention if the desired impact is to be achieved.

25.5.2 The trainees

Selection of trainees

The selection of trainees will normally be undertaken by the road organization according to established criteria. However, in practice, the criteria are often not very relevant and are not always strictly adhered to. This means that modifications may be necessary to the preferred approach in order to make it meaningful for all participants.

When selecting trainees for overseas training, special measures may be taken to ensure that the necessary qualifications have been obtained. Candidates for overseas training may be interviewed by a representative from the donor before final selection. If English is going to be the training language, the trainee candidates may be asked to pass an English examination set by the local British Council.

Type of training

The type of training will have to be adapted to the individual's absorptive capacity for theoretical and practical training.

Engineers

Engineers and other professionals are used to classroom situations, but training should never focus on lectures only (one-way communication). The trainees should be activated through discussions, and exchange of experiences should be encouraged. Classroom training should also be combined with field visits in order to illustrate the taught theoretical aspects.

Manual workers

Skilled and unskilled workers are normally not used to acquiring knowledge through classroom training or by reading books. Nor are they used to being in a learning situation for long periods. Training of manual workers should, therefore, focus on practical on-the-job training. Classroom training should take up no more than 40–50% of the time. Audio-visual aids such as films, slides, video, overheads and models should be used extensively, although this applies to all training.

Sandwich training

A training course for manual workers should have a duration of no more than 3–5 weeks. The course should be divided into modules if training of longer duration is necessary. The trainees should be sent back home between the modules to practice their newly acquired skills. This approach is called sandwich training.

25.5.3 Training programme

Targeting need

The training programme (curriculum) should be based on the identified needs of target groups. The programme should give a statement of which courses should be arranged for the different categories of personnel. Further, the programme should state the objective, the content, the form and the duration of the individual courses. Figure 25.4 shows a training programme prepared for the road organization in Sabah, Malaysia, and Figure 25.5 is a contents list for a course planned for senior field officers in Zimbabwe.

Target group

The need for training will vary considerably depending on the target group's skill and qualification levels. As stated earlier, there is an over-emphasis on higher education in many Third World countries at the expense of middle-level training of technicians and skilled workers. This results in an under-supply of the middle skill levels in general, and is a particular problem in many road organizations. Often the higher education courses tend to be very academically oriented, with the consequence that engineers lack practical skills. This adds further to the problem of not having adequate manpower at middle skill levels. It should be borne in mind that the need for skills and qualifications will be different in a national

SABAH PUBLIC WORKS DEPARTMENT
DEVELOPMENT AND STAFF TRAINING PROGRAMME

Trainees		Course Description	1984											
Group	Number		Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Engineer Trainee	1	Instructor Course A	20											
Instructors Trainees	4	Instructor Course B	20					4		10				
Overseers & O's trainees	12	Road Maintenance I		6	17	2	11							
Overseers & O's trainees	12	Road Construction I						4	30					
Technical Ass.& T.A.'s trainees	12	Road Maintenance II								13	22			
Technical Ass.& T.A.'s trainees	12	Road Construction II										15		8
Overseers & Technical Ass.	8	Surface Dressing					14			13				
Overseers & Technical Ass.	8	Asphalt Concrete									24	13		
Overseers & Technical Ass.	12	Aggregate Production								2	21			
Engineers & Eng. Ass.	20	Cost control & Admin.			15	23								
Engineers & Eng. Ass.	25	Review					15	18						11 25

Figure 25.4 Training programme from Sabah, Malaysia (source: Kampsax, Consulting Engineers).

OBJECTIVES To train the officers in basic road planning aspects.
ORGANIZATION The course programme should be elaborated by the same working groups as the course for PPOs. The working groups should be assisted by one or two Provincial Road Engineers.
DURATION: 15 days.
NUMBER OF COURSES: 10.
TOTAL NUMBER OF PARTICIPANTS: 110.
LOCATION The most conveniently situated Provincial Training Centre run by the Public Services Commission or the nearest DDF training centre.
IMPLEMENTATION The coming years in step with the amalgamation of Rural and District Councils.
COURSE CONTENT • basic mathematical concepts • elementary map reading • road standards • basic feeder road construction and maintenance • assessment of the technical viability of proposed new roads • calculation of cost estimates for construction projects • identification of road maintenance needs • benefit of feeder roads, e.g. induced agricultural production and reduced transport costs.

Figure 25.5 Description of a course for senior field officers in Zimbabwe (source: Cowiconsult, Consulting Engineers).

and centralized trunk road organization, than in a local and decentralized road organization.

In general, planning involves more than identifying a topic, a time and a date. The content and form of a course subdivides into a number of issues, some of which are listed below (ref. 7).

Course content

Course content:

- What are the precise objectives?
- What is the overall strategy?
- Will the final concepts be defined at the outset?
- Who will write the introductory outline?
- How will progressive learning be achieved?
- What will be the pattern and the structure?
- What methods and techniques will be used?
- How will maximum course member participation be achieved?
- How and when will the course tutors be briefed?
- Should a synopsis for each session be given or agreed?
- What further help will tutors need?
- How can scope be left for amendments?
- How is it possible to ensure flexibility, continuity and integration?
- Should a course tutor always be present?

- How best can the tutor act as a catalyst?
- What provision should be made for free discussion?
- Is it necessary to prepare handouts?
- Is it necessary to have worked examples?
- When and what periods should there be for private study?
- How can the optimum balance of activities be ensured?
- Is it necessary for tutors to have rehearsals? If so, when?
- What is the purpose and what should be the frequency of outside visits?
- What period should be allowed for final preparations?
- What new sessions should be included?
- What revised timings should be made?
- What open forums are necessary for course members?

Form of course

The form of the course:

- What length will the course be?
- Should it be full-time or part-time?
- Should it be residential or non-residential?
- How many sessions per day?
- What kind of sessions each day?
- What should be the balance between lecturing, instructing, demonstrations and discussions?
- What is the maximum length of unbroken lectures?
- Should the previous day's work be reviewed each morning?
- What use should be made of audio/visual aids, models and other training material?

Presentation

The training programme should be presented to the executive management of the road organization in order to get their approval and positive support. The presentation can often take the form of a one- or two-day seminar to allow for a thorough discussion and possible amendment. The presentation may also assist in identifying areas for management training to be incorporated in the overall training programme.

25.5.4 Training consultancies and contracts

Often, project-related training is carried out by international consultants under contract. In such cases, the following approach to planning is usually followed.

TOR

A broad outline and concept of the conceived training intervention is described in the terms of reference (TOR). The TOR are normally prepared by the road organization, by a donor, or may be the result of a study undertaken by a consultant. The TOR specify to which phases of the project cycle the training intervention will be related.

The training programme to be planned may comprise all the project phases. Sometimes the various phases are contracted out to different consulting firms and contractors, even though this often proves to be disruptive. The training component may be awarded to a training consulting firm, which would then be responsible for the training inputs during all project phases and have to co-operate with changing consulting firms and contractors.

Proposal

The TOR forms the basis for the consultant's preparation of the technical and financial proposal for international bidding. Since the proposal establishes the framework for the conduct of the training, already it is important at this stage to be aware of the training environment, on-going processes of structural adjustment of the road sector, in addition to the specific training requirements. The training environment and the structural adjustment may not be described very adequately in the TOR so, in order to reduce uncertainty, the firm submitting the proposal often makes an initial survey to ensure that what is offered in the proposal is compatible with the prevailing situation in the project area.

25.6 DETAILED PREPARATION

Activities

The detailed preparation of the training programme can start immediately the plan of implementation has been approved. This phase of activity includes:

- selection of trainers;
- preparation of lesson plans;
- procurement and production of training aids;
- provision of training facilities;
- planning any evaluation of the training that will be required.

Selection of trainers

Trainers will often come from a road organization's own training school, or from a local training institution, in which case they will usually have the required skills and be familiar with the training environment. However, in the case of project-related training, the project team members are frequently professionals and technicians without teaching and training experience. They will have been chosen principally in order to tap their know-how and experience. It is, therefore, important to provide them with some training themselves in order to facilitate the detailed preparation of the lessons; how to communicate with the trainees; how to prepare teaching aids, etc. In the event that the training inputs are substantial, it is advisable to arrange a course for the team members in pedagogy and instruction techniques. In most cases, it is usually best to select individuals who have a balance between technical knowledge and training skills.

Foreign trainers

Professionals and technicians who are going to train people from foreign countries need to meet certain special requirements. First of all they should be proficient in the language of instruction. This requirement seems self-evident, but is not always met. Secondly, the trainers ought to have some experience from practical work in the home country of the trainees. Otherwise it is difficult for the trainers to relate the training to the needs of the trainees. Added to this is the need to recognize that association with people from foreign cultures can involve sociological problems which may be difficult to cope with for inexperienced trainers. This can be a particular problem because great authority is normally attached to a trainer.

Lesson plans

Detailed lesson plans will have to be prepared for each course based on the training programme. A lesson plan consists of a timetable describing the content of each lesson and type of training (classroom lecture, group work, practical training, etc.). The lesson plans are often prepared by, or in co-operation with, the individual trainers who are going to deliver the training. It is important that the

SEMINAR TIME SCHEDULE 研讨课程的时间表

简例：
Legend: TW = Tim Waage 汤·华治
MA = Magne Aasen 马·阿森
BT = Bent Thagesen 宾·泰德逊
DRD= Danish Road Directorate 丹麦公路局

Monday 30/6 1986		1986年6月30日星期一
8.30 - 9.00	Opening Ceremony	开课仪式
9.00 - 9.30	Introduction to Seminar (BT)	介绍研讨课程（BT）
9.30 - 10.30	Introduction to Kampsax, Slide show (TW)	介绍金硕，放映幻灯片（TW）
10.30 - 11.00	Video: "On the Way" (TW)	"在途上"录像带（TW）
11.00 - 12.00	Introduction to DRD (MA)	介绍丹麦公路局（MA）
14.30 - 17.30	Lecture A.1 - "How to Execute a Project" (TW)	讲章A·1 — "如何执行一项工程"（TW）
Tuesday 1/7 1986		1986年7月1日星期二
8.00 - 9.30	Group work for Lecture A.1	讲章A·1的小组工作
9.30 - 12.00	Lecture A.2 - "Introduction to FIDIC Contract Documents" (TW)	讲章A·2 — "介绍国际顾问工程师联合的合同文件"（TW）
14.30 - 15.30	Lecture A.2 (continued)	讲章A·2（续）
15.30 - 17.30	Group work for Lecture A.2	讲章A·2的小组工作
Wednesday 2/7 1986		1986年7月2日星期三
8.00 - 12.00	Lecture A.3 - "Bill of Quantities and Schedules" (MA)	讲章A·3 — "工程量清单和明细表"（MA）
14.30 - 16.30	Group work for Lecture A.3	讲章A·3的小组工作
16.30 - 17.30	Lecture A.4 - "Selection of Engineer and Contractor" (TW)	讲章A·4 — "选择工程师和承包商"（TW）

Figure 25.6 Lesson plan for a training course for highway engineers in the People's Republic of China (source: Kampsax, Consulting Engineers).

lesson plans are prepared carefully in order to prevent overlap and in order to support inexperienced trainers. Figure 25.6 shows a lesson plan for a training course for highway engineers in the People's Republic of China.

Normally, training is in two parts:

Approach

- classroom
- operational.

The aim is to move as quickly as possible from the classroom situation to an operational environment, where more specific training can be undertaken.

Generally, trainees learn much more quickly by 'doing', rather than by 'watching' or 'listening'. For training in highways operations, there should be no clear distinction between training and actual operations. As trainees become familiar with the tasks to be undertaken, the level of supervision by the instructor should be gradually reduced. However, throughout the entire training period, instructors should always be on hand to ensure that methods and standards are being adhered to, and to assist in resolving any problems that might arise from time to time.

Training aids

Training aids consist of course notes and audio-visual aids. A lot of commercial training aids are available, but they are expensive and often not very suitable for the actual purpose because they are too general. Usually, it is better to produce course notes, slides, video-films, etc., that are specific to the detailed training being undertaken. This makes it possible to illustrate local conditions and local problems. It is important to reserve sufficient money for procurement and production of training aids when preparing proposals for training projects.

Course notes

Course notes should be in the language of the trainees. It is common for these to be of two types:

- paper copies of visual aids which can be used by participants during the training, and can be annotated with notes and comments;
- detailed written notes covering the technical content of the training, which can be studied after the training has been completed, and used as a source of reference.

The presentation in the course notes should be tailored to the trainees' capabilities. Course notes for professionals may contain a great deal of plain text. Course notes for slow readers may be better presented as strip cartoons, as in the example in Figure 25.7.

Visual aids

The preparation of visual aids requires some care. Good slides amplify and clarify the message, stimulate interest, and help the speaker keep 'on track'. They merit the same care in preparation as the text of a presentation. The primary consideration is legibility, since slides that cannot be read when projected lessen the impact and effectiveness of the presentation. Copies of typescript, computer outputs, graphs and illustrations from textbooks are normally too small to be seen when projected on to a screen. As a rough guide:

- no slide should contain more than 15–20 words;
- the type size on overhead projector slides should be a minimum of 7–10 mm in height.

Graphs and drawings usually need to be redrawn with increased line widths, and with captions reduced in number, simplified, and increased in size. Each slide should cover only one idea: it is better to use several simple slides rather than one more complicated one.

Training facilities

Depending on the type of training, the required training facilities may include classrooms, workshops, machines, accommodation for the trainees, etc. In some cases, the training facilities already exist and are readily available; in other cases, the facilities have to be constructed and furnished as part of the training project.

Course administration

The following provides a checklist of some of the items that need to be considered during the preparation for training courses (ref. 7):



Figure 25.7 Course notes from a training course for Road Overseas in Sabah, Malaysia (source: Kampsax, Consulting Engineers).

- ensuring initial discussion of details with all concerned;
- drafting the initial programme;
- securing line management approval of the draft;
- producing a final well-laid-out programme;
- passing final confirmation to all speakers;
- securing agreed lists of course members;
- notifying course members well in advance;
- ensuring adequate accommodation;
- arranging adequate seating, lighting and ventilation;
- giving clear instructions on contact arrangements;
- ensuring appropriate breaks and refreshments;
- securing required equipment, models and materials on time;
- arranging for any required bench space or outdoor work areas;
- arranging for required display facilities;
- making contingency arrangements for 'stand-in' sessions;
- organizing the payment of fees and expenses;
- making arrangements for meeting and introducing visiting speakers;
- coping with last minute changes to the timetable;
- checking and distributing handouts;
- arranging typing and copying;
- arrangements for any feedback to speakers;
- arrangements for any course reporting;
- arrangements for discussions of course members with supervisors.

Evaluation and tests

Evaluation procedures and tests have to be developed in advance, during the preparation stage, in order to assess the extent to which the training objectives have been met. Evaluations are normally applied to judge qualifications and attitudes. Tests are applied to judge specific skills. Diplomas and certificates can be provided to successful candidates. Evaluation procedures may also be developed in order to get the trainees' own impression of the quality of the training provided.

A training evaluation pack needs to be developed for each session. This will consist of both test material and evaluation criteria. Simple test material should be provided that can be used during and at the end of a session to test comprehension. This can be written, but is normally oral, for use in an interactive manner. Further evaluation material is normally provided for use at regular intervals after the formal training has been completed to ensure that principles and methods are still understood, and to identify any needs for retraining.

25.7 IMPLEMENTATION

25.7.1 Communication

Modes of communication

Communication is the essence of effective training. Without effective communication, no instructions can be given or received and nothing can be achieved. There are several modes of communication, each with its own set of mechanisms and channels:

- oral

- visual
- written.

In the context of highways training, the oral and visual modes are those most used, although written communications also have a role.

Mechanisms

To have effective communication, it is necessary to understand the basic mechanisms. Communication is more than just 'telling'; it has three components:

- source
- channel
- receiver.

In order for information to pass from the source (trainer) to the receiver (trainee), the following things must take place:

- first, the trainer must know, with some precision, what is the message to be conveyed;
- second, the message which consists of 'internal' thoughts must be 'externalized' through the use of both verbal language and body language;
- third, the trainee must have excellent reception of the signals being transmitted;
- fourth, the trainee must integrate the numerous signals received and turn them into understandable thought and action.

A breakdown or inadequacy in any of these steps will prevent effective communication taking place.

Success rates

This may sound theoretical, but effective communication is more difficult to achieve than most people realize, and success rates are typically less than 25%. This means that the odds are three to one against communication being successful, with the message received being the one that the trainer meant to convey. The quality of reception and understanding will be conditioned by the trainee's:

- prior knowledge and basic assumptions;
- attitude;
- frame of mind.

Feedback

It is important that the person at the source is aware of this and reflects these constraints when framing the way that the information is transmitted. In order to check that communication has taken place, and that information has been received, understood and produced the required reaction, some form of feedback is advisable. Without effective feedback, the effectiveness of future actions may be hampered because of a lack of appreciation by the trainer for the reasons for communication being ineffective in the past. It is, therefore, crucial that, for training to be effective, the trainer understands the basic mechanisms of communication.

25.7.2 Instruction

Examples

It is beyond the scope of this book to provide a detailed training manual. However, the following are some examples of where inexperienced trainers offend the most basic rules of good instruction:

- The trainer is not properly prepared; the fact that a good instructor never reads direct from a manuscript sometimes leads inexperienced instructors to believe

that a lecture does not need to be prepared carefully – this is wrong: even experienced instructors attach much importance to the preparation of lectures.
- The trainer speaks to the blackboard or to the screen instead of facing the audience; the trainer should always be aware of who he is addressing.
- The size of the lettering on slides is so small that it cannot be read from the back of the room.
- The trainer gives too much and too detailed verbal information; lectures are only suitable for presenting a general view of a subject and for emphasizing a few essential messages.
- The trainer focuses too much on one-way communication; discussions, exercises, group work, etc., should play an important part in training; however, preparation of this type of training requires experience and is time-consuming.
- The duration of classes is too long; it has been demonstrated that the benefit from a lecture decreases rapidly with the duration of the lecture: a lecture should never last more than 30–40 minutes without a break.

25.7.3 Monitoring and feedback

Monitoring

The main concern when implementing a training programme is to ensure that the quality and content is in accordance with the training needs assessment and objectives. Unexpected problems may occur, which call for revision of curricula, evaluation criteria, and the like. It is important that the implementation process is monitored adequately by the training manager and that modifications are made immediately, if and when required. Evaluation and tests should be spaced to ensure that established quantitative and qualitative standards are met throughout the training process.

Feedback

The evaluation made at the end of the training programme will reveal the extent to which objectives have been met. Where objectives have not been met, feedback mechanisms should ensure that corrections are being made for succeeding training activities to avoid repetition of mistakes. However, the progress monitoring should prevent any serious problems arising.

Post-evaluation

Even in those cases where the training objectives have been met, this does not necessarily mean that the corporate performance requirements will also be achieved: the road organization may not draw on the newly acquired qualifications of the trainees; some trainees may not want to share their new knowledge with others. Issues such as these can only be revealed after some time when the trainees have been working in their assigned positions. A post-evaluation is seldom made, but it can provide a rich source of valuable information which could be utilized either in the organization's permanent training set-up, or in the design of succeeding training programmes.

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