GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH

LOCAL GOVERNMENT ENGINEERING DEPARTMENT

ROAD PAVEMENT DESIGN MANUAL

Prepared by
The Technical Working Group
LGED

A C R O N Y M

AADT	-	**Annual Average Daily Traffic**
AASHTO	-	**American Association of State Highway and Transportation Officials**
AIV	-	**Aggregate Impact Value**
ASTM	-	**American Society for Testing & Materials**
ASS	-	**Aggregate-Sand-Soil**
BC	-	**Bituminous Carpeting**
BS	-	**British Standard**
BRRL	-	**Bangladesh Road Research Laboratory**
C	-	**Cumulative number of commercial vehicle**
CB	-	**Cement or lime stabilised road base**
CBR	-	**California Bearing Ratio**
CS	-	**Cement or lime stabilised subbase**
DBST	-	**Double Bituminous Surface Treatment**
ESA	-	**Equivalent Standard Axle**
ETR	-	**Environmental Trial Road**
FM	-	**Finness Modulus**
FRB	-	**Feeder Road Type-B**
GB	-	**Granular roadbase**
GC	-	**Granular Capping Layer**
GDP	-	**Gross Domestic Product**
GI	-	**Group Index**
GS	-	**Granular Subbase**
GVW	-	**Gross Vehicle Weight in Tonnes**
HBB	-	**Herring-Bone-Bond**
IRC	-	**Indian Road Congress**

LAA	-	**Los Angles Abrasion**
LGED	-	**Local Government Engineering Department**
LL	-	**Liquid Limit**
MDD	-	**Maximum Dry Density**
OMC	-	**Optimum Moisture Content**
PGI	-	**Partial Group Index**
PI	-	**Plasticity Index**
PM	-	**Plasticity Modulus**
PP	-	**Plasticity Product**
RB	-	**Bituminous Road Base**
RCTP	-	**Road Construction Trial Project**
RDP	-	**Rural Development Project**
RHD	-	**Roads & Highways Department**
RR	-	**Reinforced Road**
S	-	**Cumulative Standard Axle**
SB	-	**Subbase**
SS	-	**Sand-Soil**
T	-	**Traffic Classes**
TRRL	-	**Transport & Road Research Laboratory**
UK	-	**United Kingdom**
WBM	-	**Water Bound Macadam**

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1.0 INTRODUCTION

The road system in Bangladesh is classified into the following seven categories :

	Category	Length (Km)	Definition
1	National Highway (NH)	3,163	Connecting national capital with divisional headquarters, district headquarters, port cities and international highways
2	Regional Highway (RH)	2,911	Connecting different regions with each other which are not connected by the national highways
3	Feeder Road Type-A (FRA)	9,996	Connecting Thana Headquarters to the arterial network
4	Feeder Road Type-B (FRB)	17,058	Connecting growth centers to the RHD network (FRA or arterial road) or to the Thana Headquarters
5	Rural Road Category-1 (R_1)	56,136	Connecting Union Headquarters/local markets with the Thana Headquarters or road system
6	Rural Road Category-2 (R_2)	48,874	Connecting villages and farms to local markets/Union Headquarters
7	Rural Road Category-3 (R_3)	61,286	Roads within villages

The National Highways, Regional Highways and Feeder Roads Type-A (FRA) are the responsibility of the Roads and Highways Department (RHD) while Feeder Roads Type-B (FRB) and Rural Roads R_1, R_2 and R_3 come under the jurisdiction of the Local Government Engineering Department (LGED).

The roads under the jurisdiction of LGED play a significant role to serve the vast rural Bangladesh. These provide local access to farms, social and welfare institutions, village facilities as well as to markets where buyers and sellers assemble to trade products of agriculture and rural industries and other consumer goods. They also provide access to modern inputs to agriculture, such as, fertilizer and seeds as well as to local places of employment.

Until recently, the common practice for FRB and rural road construction was to use Herring Bone Bond (HBB) or Water Bound Macadam (WBM) for pavements. Following some problems with the quality and wear and tear of HBB, LGED is favouring WBM/AS at present and in order to protect the investment, the same is being covered with a bituminous surface layer.

Many factors contribute to the poor condition of FBB and rural roads. Most of the roads were built with labour-intensive technology under the Food for Work (FFW) Programme. Many of the embankments were built with poor compaction and no structures (bridges/ culverts) were provided. The poor nature of the soils available to build embankments also contributes to poor conditions. Suitable construction materials are rare, even for embank-ments, not to mention the higher-grade material needed for substantial surfacing and traffic bearing layers. These requirements make the cost of building roads in Bangladesh some of the highest in the world and also demand that greater than normal attention be devoted to maintenance.

LGED prepared the Earth Work Manual for construction of road/embankment and the Design Manual for construction of culverts/small bridges and sluices in the early 1980s. The Road Structures Manual was prepared by LGED in 1989 for construction of bridges/culverts and the same was revised in 1997. Moreover, LGED prepared a Manual on Prestressed Concrete for construction of large bridges in 1996.

The rural infrastructure development projects implemented so far by LGED included, construction of FRB and rural roads along with bridges/culverts as the major compo-nents. In absence of a Pavement Design Manual, different projects of LGED introduced different set of standards for their individual projects.

In order to unify the system and to make the best utilization of the available local materials, it was decided by LGED to prepare a Pavement Design Manual. A Technical Working Group headed by an Additional Chief Engineer of LGED was formed for pre-paration of the Manual. The composition and the Terms of Reference (TOR) of the Working Group are given in Appendix-H.

2.0 EARTH WORKS COMPONENTS

2.1 Earth Work

For the construction of any road irrespective of its category, either we need to fill the embankment to raise the road above flood level or cut it down to bring it to its formation level.

2.1.1 Fill

The topographic condition of Bangladesh is such that most of the road constructions require embankments rather than cuttings. Almost 85 percent area of our country is a major deltaic region and it is underlain by deltaic and alluvial deposits of the Ganges, the Brahmaputra/Jamuna and the Meghna rivers systems. Fortunately this deltaic and alluvial soil is most suitable for agricultural purpose but unfortunately it is not that much suitable as road construction material. Moreover, when a high embankment is needed to cross a gully or at an approach to water crossing, the embankment itself imposes a substantial amount of load on the underlying soil and a settlement should always be expected for such type of underlying soil. Situation is more aggravated when organic soil is encountered during construction and it is very much frequent in the area of southern, southwestern and northwestern part of the country. In spite of low quality of soil encountered frequently, embankment of the road should be constructed with the locally available soil to avoid long hauling distance in order to reduce the excessive construction cost. But the selection of source and borrowpits should be done carefully by an experienced and qualified person because the quality of material may vary even the sources are not very far from each other. But in any case, organic materials like peat, black cotton soil and mug must not be used as these materials are highly compressible. Also pure silt is better to avoid.

2.1.2 Soil Characteristics in Bangladesh

East, north and north eastern side of Bangladesh is a peripheral hilly region being made up of neogene and palaeogene sediments whereas its adjoining recent piedmont deposits and the vast distinctive tracts of residual deposits popularly known as the Madhupur and Barind clay residuum. In the hilly region, since the sediments derived are related to the various lithological units, the composition of the units plays direct role in determining the nature of the derived sediment deposits. Since shells, clay stones, silt stones and sand stones make up most of the formations of the constituent materials they are also dominant among the materials derived from the formations of the hilly region.

Particularly in the north and northeastern part of the country, vast tracts of land is covered either by extensive or isolated remnants of clay residium with reddish to red brown mottled clay containing little amount of silt and sand. These tracts cover extensive areas in the central and northern zones of Dhaka, north-eastern region in Sylhet Division and in the central and southern areas of Rajshahi.

Most part of these low-lying deltaic country flood plains is a result of an intricate network of rivers with poor channel gradients and rapid siltation. Heavy and high intensity of seasonal precipitation generates a tremendous amount of surface run-off in upper region of the rivers causing the silted channel to over flow which, in turn, contribute to recurrent flooding. When the rivers move from upstream to the down, the gradient of the streams become flatter and flatter consequently the velocity of flow becomes slower and slower. As a result coarser sediment like sand, deposits at upstream and finer particles like silt and clay wash down to the flooded plains and basins forming clay and silt mantles. The basic characteristics of the flood plain alluvium of the major rivers are as follow:

- The flood plain sediments of the Brahmaputra/Jamuna and the Meghna have a high silt content,
- The Teesta and western part of the Ganges flood plain sediments are extensively sandy,
- The lower part of the Ganges/Padma flood plain sediments has clay content and partially calcareous.

When we consider that the sediment being deposited will be used in future road construction, it is disappointing that the recent sediments are typically dark gray, loosely consolidated with a high water content and also containing variable organic matter. These unconsolidated sediments are far from homogeneous in age, texture and mineralogy. They are being deposited in piedmont plains, meander flood plains, tidal flood plains and estuarine flood plains which have been uplifted or downwrapped by subsequent earth movement.

Some of the piedmont deposits occur in the most elevated parts of the country at the foot of the northern and eastern hilly areas. The deposits are mainly sandy with occasional pebbles present in them. These are the results of active torrential erosion in mountains and deposition by river or streams as alluvial and colluvial materials. The composition of such materials range from boulders, shingles to gravels mixed with a coarse to fine sandy matrix.

It is found that most of the southern areas of the country namely, Khulna, Patuakhali, Barisal, Noakhali and Comilla, the alluvial deposits are finely stratified to great depths as estuarine flood plain deposits. They are distinctively silty materials with only a relatively shallow depth of soil formation. The tidal flood plain deposits, in a very limited term, occupy only the southern part of the Ganges/Padma flood plain and parts of the Chittagong coastal plain. Clays are their main components with buried peat layers within some closed basins and presence of silts which are calcareous at and around the Ganges distributaries. The similar tidal clay plains occupy most of the offshore islands.

2.1.3 Side Slope Selection

The most neglected part of our road construction is the construction of side slopes and in some cases the shoulder. Although the shoulders and the side slopes are done once at the initial construction stage, no proper care is taken during the maintenance period. As a result, not only traffic flow capacity of a road is reduced but also threatens to failure of the road as a whole in many occasions. Following steps are required to be followed in order to obtain a stable embankment.

- The first step in obtaining a stable roadway embankment is the careful selection of alignment of the road in respect of topography and drainage
- The second step is the proper design of the embankment and cut slopes
- The third step is the use of proper construction materials and techniques to obtain the required stability.

In fact the stability of embankment basically depends upon the shear resistance offered by the soil to external forces. On the other hand, soil shearing resistance is a function of the apparent cohesion of the soil and the angle of internal friction, which are further dependent upon the moisture content and dry density of the soil at the time of loading. However, elaborate tests involved in determining these properties of soil may not be needed for ordinary road works. But the following factors should however be kept in mind while choosing the side slopes:

(i) The type of soil of which the slope is made of
(ii) The climatic conditions, specially the amount of annual rainfall in the area
(iii) The adequacy of drainage
(iv) The nature and characteristics of the surface layer

A particular stable slope made out of a soil when it is dry may not remain stable when wet and vice-versa. This is particularly true at least for pure clay and pure silt. It is further essential to distinguish between virgin soils in cut and the made-up soils in embankments. In practice, it may sometimes be possible to measure the angle of repose of the soil in field but such measurement shall be subject to the exercise of judgement in respect of weathering, moisture content of the soil, construction procedure and drainage.

A relatively rational approach to the problem of proper slope design for embankments has been presented by the Highway Research Board in 1952. Accordingly, the soils are classified as per American Association of State Highway and Transportation Officials and also in accordance with the height of fill and the exposure to the climatic conditions in the area. Based on these information, the proper slope for the embankment may be taken from **Table 2.1**. It may be observed from this table, the recommended slopes have also been corrected for degree of compaction in the field.

TABLE 2.1

Recommended Slopes and Recommended Minimum Requirements for Compaction of Embankments

Class of soil as per AASHTO	Not subject to inundation			Subject to periodic inundation		
	Height of fill: (metre)	Slope	Compaction:% of max. density as per std.proc.	Height of fill: (metre)	Slope	Compaction:% of max. density as per std. proctor.
A-1	Not critical	1-1/2:1	95+	Not critical	2:1	95+
A-3	Not critical	1-1/2:1	100+	Not critical	2:1	100+
A-2-4	Less than 16	2:1	95+	Less than 13	3:1	95+
A-2-5	Less than 16	2:1	99+	3 to 16	3:1	95 to 100
A-4	Less than 16	2:1	95+	Less than 16	3:1	95 to 100
A-5	Less than 16	2:1	95+	Less than 16	3:1	95 to 100
A-6	Less than 16	2:1	95+	Less than 16	3:1	95 to 100
A-7	Less than 16	2:1	95+	Less than 16	3:1	95 to 100

- For class of soil please see Appendix ‘A’.

- Recommendation for this condition depends upon the height of fill. For lower heights of the embankment the slope may be adjusted to fit the local situations. In case of higher embankments construction of Berm may be considered. Higher the fills of the order 11.00 metre to 15.00 metre should be compacted to 100%, at least for those parts of the fills, that are subjected to periodic inundation.

The slope can be a little bit steeper in case of cut. But it must be based on experience, choice and judgement. In practical consideration, the design of slope could better be done in such a way that the driver sitting in his seat in the vehicle is able to see the bottom of the slope where it meets the natural ground. This develops the confidence in the driver to come closer to the edge of the embankment whenever he requires to allow another vehicle to pass and thus reduces the chance of accident as well as increases the capacity of the road.

While the existing embankments need to be widen to accommodate the new construction, it should be a normal practice to cut horizontal benches into the slope to simplify construction and to help key the existing embankment to the newly widened portion. Embankment widening procedure by cutting bench is shown in **Figure 2-1**.

2.1.4 Slope Protection

The most common practice in the country is to protect the slope from erosion by growing a suitable type of vegetative cover like grass at the top. Most of the soils used in slope construction contain some degree of cohesive material which is conducive to such a treatment. But purely cohesionless soils may not easily support vegetation. For such type of soil, a cover of 25 to 30 cm layer of clay soil may be uniformly laid and stabilized before turfing with grass.

Some of the slope protection techniques are given below:

(i) Sowing seeds for vegetable/grass turfing,
(ii) Transplantation of ready-made turfs of grass,
(iii) Use of straw with cowdung as mulch,
(iv) Vegetative turfing with jute netting or coir netting,
(v) Loose stone riprap (this technique may be used in the hilly areas)
(vi) Grouted stone riprap (where slopes are steep and stones are readily available)
(vii) Cement concrete block riprap,
(viii) Reinforced concrete slope protection,
(ix) Brick mattressing
(x) Sand-cement gunny bag riprap

Cement concrete block riprap and brick mattressing are in common use at high approaches of the bridges or where the wave action is a major threat to the embankment.

2.1.5 Cuttings

In the hilly areas of the country where the alignment of the road passes through the hills and vallies alternately, the designer needs to be careful to balance cutting with filling to minimize wastage and reduce hauling distance in order to make the construction cost-effective. Machine or manual excavation method is generally employed for earth and soft rock cuttings. But for the very hard rock excavation, blasting may sometimes be required to make the job faster. Proper planning in the initial stage is essential to optimise the use of machinery and materials. The drainage problem is always associated with cuttings which demands a careful consideration although the longitudinal side ditches may be universally used for the purpose.

Cutting through sound rock can often stand at or near vertical, but in weathered rock or soil the conditions are more unstable. Instability is usually caused by an accumulation of water in the soil and slips occur when this accumulation of water reduces the natural cohesion of the soil and increases its weight. Thus the design and construction of the road should always promote the rapid and safe movement of water from the area above the road to the area below. Under no circumstances should the road impede the flow of water internally or externally or the road embankment stands as a barrier to water movement.

In rock cutting, the side slopes may be taken from 1/4 to 1 horizontal to 1 vertical depending upon the rock quality designation and climatic condition.

The subgrade in cutting requires special attention to meet the requirement of the specified subgrade properties. Most of the cases the soil properties are prescribed in the specification and the minimum requirements are outlined. As such during construction when cutting reaches at the top level of subgrade, the soil sample should be collected and tested in the laboratory whether the soil satisfies the required properties or not. If the soil is found to meet the specification, the area should be scarified up to the required depth as stated in the specifications and the moisture content should be brought to the optimum level either by aeration or by spraying water as the case may be and compacted up to the specified degree of compaction. If the soil in place fails to satisfy the required properties, the soil in question should be removed (at least 600mm from the top) and replaced with specified soil and the subgrade should be prepared accordingly.

3.0 HYDROLOGY

3.1 High Flood Level Determination

In determining HFL along the road several points must be considered and applied depending on the type of situation as described below :-

(i). Very often it is not possible to get hydrological data of the desired location. However, data are available for places which are hydraulically connected to the desired site.

(ii) The river or its tributaries which are affecting the HFL along a particular road should be determined after a careful study of the topographical map. The direction of flow should also be studied. Data on river stage of two nearby gauge stations of the road should be analyzed by the frequency distribution method. HFL of 10 or 20 yr return period along the road is then determined by means of co-relation and interpolation.

(iii) Field verification of the assessed HFL should be made by interviewing local people, by identifying water marks on the surface of tree trunks or on the walls and foundation of structures (e.g. buildings, bridges, culverts etc.). Abnormal flood levels experienced in 1987/1988/1998 should not be taken into consideration.

(iv) Observation of physical features should also be considered determining HFL. Because the plinth level of a house is generally constructed above HFL. Existing bridges and culverts on an alignment as well as level of railway bridge near by may also be used as indicators for determining HFL.

(v) Marking of HFL on permanent pillars by BWDB indicate the maximum level in terms to PWD datum. These markings should be used as a tool in adjusting HFL.

(vi) Land use pattern should also be studied in determining or adjusting the HFL. For example, paddy/jute fields are generally flooded but fruit and vegetables gardens are normally above the flood level.

4.0 FLEXIBLE PAVEMENT DESIGN

In a broad sense of the term, pavements can be divided into two types, namely, rigid and flexible. Rigid pavement as the name implies is a portland cement concrete slab acting as wearing surface of the road. All other types of pavement than rigid can traditionally be classified as flexible. The widely accepted definition of a flexible pavement is that, “a flexible pavement is a uniform or composite structure that maintains an intimate contact with and distributes loads to the subgrade by mechanical interlocking of aggregates, particle friction and cohesion for developing stability”. Thus, the classical flexible pavement includes primarily those pavements which are composed of a series of granular layers (bituminous or non bituminous) by a relatively thin layer of wearing surface made of high quality materials. A flexible pavement may be composed of a single layer or a series of layers depending mainly on traffic volume. It may happen that on a heavy duty pavements like national highways or super express highways, multiple bituminous layers might be used. Two typical cross-sections, one for light duty pavement and the other for heavy duty pavement are shown in **Figure 4-1** and **Figure 4-2** respectively.

It may be pointed out that certain pavements like very thick asphalt surface, neither they behave as rigid pavement nor they behave as perfect flexible pavement. Similar is the case for cement-grouted roads. Both of these types of pavements may be termed as semi rigid pavements depending on their behavior. However, for the convenience of presentation, very thick bituminous wearing surface is also classified as flexible pavement.

The higher quality of materials require for the upper layer from the next lower one and eventually the cost of construction of the upper layers increases gradually.

Normally, the wearing surface may vary in thickness from 25mm or even less in the case of Bituminous Surface Treatment to 150mm or more in the case of asphalt concrete used for heavy duty traffic. The wearing surface must be of good quality for withstanding the wear and abrasive effects of moving wheels and must possess sufficient strength to prevent it from shoving, ravelling and rutting under traffic loads. In addition, most importantly, it must serve a useful purpose in preventing the infiltration of surface water into the base, sub-base and subgrade directly from top. The basic concept of the flexible pavement is that the combined thickness of sub-base, base and wearing surface must be sufficient to reduce the action of stresses developed from wheel load before reaching to the subgrade to a level that are within bearable limit not to cause excessive deformation or displacement of the subgrade layer.

The fundamental differences between flexible pavement and rigid pavement are presented below:

i) Flexible pavements certainly yield to over stresses those occur due to frequent or occasional wheel load passing over it as it being heavier than that load for which it was designed. As a result a depression becomes visible on the surface for the flexible pavement. On the other hand, rigid pavement under heavier loads than that of designed one, produce ruptures showing the marks of cracking on the surface to help relieve the stresses induced in it.

ii) If a localized variation of strength exists in the subgrade, a flexible pavement can adjust itself to the irregularities by undergoing to differential settlements while a rigid pavement cannot adjust it without cracking.

iii) Temperature changes due to change in atmospheric conditions do not produce stresses or very negligible stresses for certain conditions in the flexible pavements but the variation of temperature induce heavy stresses in the rigid pavement. As such, contraction and expansion joints are a must for the rigid pavements.

iv) Stage construction for flexible pavement is very much possible where as it does not so for the rigid pavement.

v) So long the damage has been caused within itself, flexible pavement has the self healing properties but the rigid pavement does not have that at all. If any deformation occurs in flexible pavement within itself due to heavier load than expected, it can recover it to some extent with time under favorable weather conditions. But any damage is permanent for the rigid pavement.

vi) The fundamental difference between the rigid and flexible pavement is on principle how they distribute the wheel load on the subgrade. A flexible pavement, in normal practice, is composed of a series of layers with higher quality of materials at higher layers, functions mainly in a manner by way of load dispersal through the component layer depending upon their load spreading capacities and characteristics. As a result, a gradual reduction of stresses occurs until to a level before it reaches to the subgrade so that it does not exceed the limit of the safe bearing capacity of the subgrade. On the contrary, a rigid pavement by virtue of its rigidity and high elastic modulus has a chance to distribute the load over a relatively wider area of subgrade. A substantial portion of structural capacity is thus provided by the concrete slab itself. Moreover, the slab can cover the gap by bridge action, if, in any case, intimate contact of the slab does not occur with the entire area of the subgrade underneath. Therefore, the flexural strength of concrete is a major factor in designing a rigid pavement.

vii) Rigid pavement construction is the best solution for submersible road where road constructed at low hieght passing through haor areas.

viii) In the hilly areas or for crossing a valley it is wise to construct Irish-crossing or causeway with rigid pavement as it has got some definite advantages over flexible pavement because of their different nature of performance.

ix) The initial construction cost of rigid pavement is higher than the flexible pavement.

x) The maintenance cost of rigid pavement is much less than the flexible pavement.

4.1 Elements Effecting Flexible Pavement Design

A general study of the available design methods and research documents reveals that there are five universally recognized elements which demand due consideration in arriving at a rational design. These are the following :

i) Estimating the volume and character of traffic that will use the road over selected period of design life,

ii) Assessing the strength and characteristics of subgrade soil over which the road to be constructed,

iii) Considering the climate or environmental condition of the area that should follow the alignment of the road,

iv) Considering the effect of moisture that will remain present under the completed pavement and over all drainage condition of the area,

v) Selecting the most economical combination of pavement materials and layer thickness that will provide satisfactory service over the design life of the pavement.

Other than those mentioned above, some other minor aspects of pavement design should be given special emphasis in designing road pavements in the most tropical countries like ours.

- The severe weathering conditions imposed on exposed bituminous surfacing materials by tropical climates and the implication of this for the design of such surfacings,

- Another aspect of design is the inter-relationship between design and maintenance. If an appropriate level of maintenance cannot be assumed or ensured, it is not possible for the pavement to carry the anticipated or forecsated traffic loading without incurring high costs to vehicle users through increased road deterioration,

- One of the most important and most common aspect of design is the uncontrolled high axle loads and tyre pressures that most frequently being noticed in our country.

4.2 Design Methods

From extensive investigations and research works since long, a lot of flexible pavement design methods have been developed and being implemented throughout the world. Some of the methods are mentioned below:

(1) Indian Road Congress (IRC) method,

(2) Wyoming method,

(3) Road Note 31 method (A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical Countries),

(4) California Bearing Ratio method,

(5) Group Index Method

Of all the methods mentioned above, for flexible pavement design, we will follow the **IRC method** as this is simple and easy to follow. Method related to **Road Note 31** will be Annexed.

4.2.1 IRC Method :

IRC Method is widely used in India and in neighbouring countries for the flexible pavement design. The method may be divided into two broad categories :

i) Standard Axles Method
ii) CBR Method

The factors affecting the flexible pavement design which are characteristics of soil, traffic condition, moisture, climatic condition, stress distribution etc. have already been discussed.

4.2.1.1 Standard Axles method :

In Standard Axles method traffic is defined in terms of the cumulative number of standard axles (8160 kg) to be carried during the design life of the road. It is well recognised that the structural damage caused by a vehicle depends on the axle load it imposes on the road, and the equivalent axle load concept is the best method available, for design purposes, to handle the large spectrum of axle loads actually applied to a pavement.

The design curves relating pavement thickness to the cumulative number of standard axles to be carried for different sub-grade strength values are given in **Figure 4-3**. The subgrade strength is assessed in terms of the CBR value of the sungrade soil. The thickness deduced from **Figure 4-3** are total thickness and consists of various combinations of bituminous surfacing and granular base and sub-base thicknesses. The recommended minimum thicknesses and compositions of component layers for new constructions are given in Thickness Combination Block, **Figure 4-4** and **Table 4.1**.

BITUMINOUS SURFACING	TOTAL THICKNESS 'T'
GRANULAR BASE	FROM THICKNESS CHART **(Fig. 4-3)**
GRANULAR SUB-BASE	

Fig. 4-4 Thickness combination Block

TABLE 4.1 STRUCTURAL SECTION

Cumulated Standard axles	Minimum thickness of component layers compacted thickness (mm)		
	Surfacing (X)	Base (Y)	Sub-base(Z)
0.5 M	20 mm PC/2-Coat SD	150	(T-150) Minimum thickness 100 mm on subgrades of CBR less than 20%
0.5-2 M	20 mm PC/MS	225	(T-225) Minimum thickness 150 mm on subgrades of CBR less than 20%
2-5 M	20 mm PC//MS/SDC + 50 mm/75 mm BM	250	(T-300/325) Minimum thickness 150 mm on subgrades of CBR less than 30%
5-10 M	25 mm SDC/AC + 60 to 80 mm DBM	250	(T-335 to 355) Minimum thickness 150 mm on subgrades of CBR less than 30%
10-15 M	40 mm AC + 65 to 80 mm DBM	250	(T- 355 to 370) Minimum thickness 150 mm on subgrades of CBR less than 30%
15-20 M	40 mm AC + 80 to 100 mm DBM	250	(T-370 to 390) Minimum thickness 150 mm on subgrades of CBR less than 30%
20-30 M	40 mm AC + 100 to 115 mm DBM	250	(T-390 to 405) Minimum thickness 150 mm on subgrades of CBR less than 30%

SD - Surface dressing to the MOST Specification/IRC Standards
PC - Premix Carpet -do-
MS - Mix Seal Surfacing to the MOST Specification
SDC - Semi-dens Carpet -do-
AC - Asphaltic Concrete -do-
BM - Bituminous Macadam Binder Course to the MOST Specification
DBM - Dense Bituminous Macadam Binder Course

Note :
(i) If the CBR of the subgrade is more than the minimum requirement for the sub-base, no sub-base is required.
(ii) Binder course of thickness more than 80 mm should be laid in two layers.

4.2.1.1.1 Traffic :

Traffic is the key factor that affects the thickness of the pavement design. For the purpose of structural design only the number of commercial vehicles of laden weight of 3 tonnes or more and their axle-loading will considered. To obtain a realistic estimate of design traffic due consideration should be given to the existing traffic or that anticipated in the case of new constructions, possible changes in road network and land use of the area served, the probable growth of traffic and design life.

Estimate of the initial daily average traffic flow both way for any road should normally be based on 7-day 24-hour calssified traffic counts. However, in exceptional cases where this information is not available 3-day count including weekly hat day could be used. In cases of new roads traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.

4.2.1.1.2 Computation of traffic

The design traffic is considered in terms of the cumulative number of standard axles (in the lane carrying maximum traffic in case of double lane) to be carried during the design life of the road. Its computation involves estimates of the initial volume of commercial vehicles per day, lateral distribution of traffic, the growth rate, the design life in years and the vehicles damage factor (number of standard axle per commercial vehicle) to convert commercial vehicles to standard axles.

The following equation may be used to make the required calculation :

$$N_s = \frac{365xA[(1+r)^x - 1]}{r} \text{ x } F$$

Where

N_s = The cumulative number of standard axles to be catered for in the design

A = Initial traffic, in the year of completion of construction, in terms of the number of commercial vehicles per day

r = Annual growth rate of commercial traffic

x = Design life in years

F = Vehicle damage factor (number of standard axles per commercial vehicle)

4.2.1.1.3 Vehicle damage factor, F

The vehicle damage factor is a multiplier for converting the number of commercial vehicles of different axle loads to the number of standard axle-load repetitions. The vehicle damage factor is arrived at from axle-load surveys on typical road section so as to cover various influencing factors such as : traffic mix, type of transportation, type of commodities carried, time of the year, terrain, road condition and degree of enforcement. The AASHO axle-load equivalence factors may be used to convert the axle load spectrum to an equivalent number of standard axles. For designing a new road pavement or strengthening an existing road pavement, the vehicle damage factor should be arrived at carefully by using the relevant available data. Some indicative values of vehicle damage factors are given below :

TABLE 4.2 INDICATIVE VDF VALUES

Initial traffic intensity in terms of number of commercial vehicles/day	Terrain	VDF values (standard axles of 8.16 tonnes per commercial vehicle)		
		Unsurfaced	Thin bituminous surfacing	Thick bituminous surfacing
Less than 150	Hilly Rolling Plain	0.5 1.5 2.0	0.75 1.75 2.25	
150-1500	Hilly Rolling Plain		1.0 2.0 2.5	1.25 2.25 2.75
More than 1500	Hilly Rolling Plain		1.25 2.25 2.75	1.5 2.5 3.0

Where sufficient information is not available, the tentative indicative values of vehicle damage factor as given in **Table 4.2** may be used. These may be judiciously modified for any special conditions with regard to traffic mix, type of transportation, etc. The validity of the value chosen may be checked after the pavement has been put to use, so that the warranted corrective steps can be undertaken.

4.2.1.1.4 Growth Rate:

An estimate of likely growth rate can be obtained by studying the past trends in traffic growth. If adequate data is not available, it is recommended that an average value of 7.5 per cent may be adopted for rural routes.

4.2.1.2 CBR method

Where data is not available to adopt the equivalent axle load concept, the CBR method which considers traffic in terms of commercial vehicles per day may be used provided the design traffic is not more than 1500 commercial vehicles per day. The CBR curves updated for 10.2 tonnes single axle legal limit presently in force are recommended for design in **Figure 4-5**. The thickness of different layers of sub-base, base and surfacing can be determined by repeated use of these curves and duly taking into account the minimum thickness and compositional requirements. The following approximate traffic conversions may be used to decide the minimum thickness and composition of various layers :

Traffic range	Adopt minimum layer thickness and composition applicable to
i) Design traffic upto 150 cv/day (CBR curve A,B,C)	Upto 0.5 million standard axles (msa)
ii) Design traffic 150-450 cv/day (CBR curve D)	0.5 to 2 msa
iii) Design traffic 450-1500 cv/day (CBR curve E)	2 to 4 msa

4.2.1.2.1 Axle Load

Along with other factors, the maximum wheel load values control the pavement thickness and the tyre pressure controls the wearing surface. Since the design method is empirical, it is not essentially related to any particular value of axle load or wheel load repeatations. But for the purpose of Design, curves should be considered operative upon single axle (dual wheel assembly, i.e., 4 wheels mounted on the axle) loads of 8200 kg. Thus, the load transmitted to a single wheel stands at 8200 kg ÷ 4 =2050 kg, which is considered to be the upper most limit of this CBR design. Arrangement of the axles like single and Tandem is shown in **Figure 4-6(A).**

The duel wheels on each side of the axle act independently just at contact surface of the road upto certain depth, approximately half of the clear spacing between two wheels, i.e., d/2 as shown in **Figure 4-6(B).** The individual wheel load, therefore, ceases to act independently and the stresses developed by the wheels just below d/2 depth act together combinedly. Stresses, at this stage, though overlap but reduce to lower intensity as the depth increases (spread to the larger area). Analytical works indicate that the overlapping stresses become negligible at a depth of twice the c/c distance of the dual wheels as shown in **Figure 4-6(B).**

4.2.1.2.2 Traffic Classification

Light vehicles are considered not to be quite injurious to the pavement. Thus for the pavement design purpose, the heavy vehicles are considered only. As such, the commercial vehicles of laden weight of 3 tons or more are considered for the use of the CBR curve in **Figure 4-5**. The number of such vehicles plying on the road in both directions per day is estimated and classified in **Table 4.3.**

TABLE 4.3

Traffic Classification	
Traffic	CBR Design Curve Applicable
0-15	A
15-45	B
45-150	C
150-450	D
450-1500	E
1500-4500	F
Exceeding 4500	G

4.2.1.2.3 Traffic Projection

The traffic classification shown in **Table 4.3** is not done on the basis of the number of vehicles likely to ply on to the new road immediately after its construction. It is normally done after appropriately predicting the number of vehicles that would use the road at end of the design period.

Estimation of initial daily traffic should normally be used on 7 days - 24 hours classified traffic counts. However, in exceptional cases where 7 day count is not available, 3 days count could be used. Traffic will obviously increase significantly with the completion of a new road. As such, traffic projection is required to be made for the design period of the road. The following projection formula may be used for predicting traffic at the end of the design period.

$$A = P(1+r)^{n+N}$$

Where, A = Number of commercial vehicles per day in both directions

P = Number of commercial vehicles per day at present or at the time of last count.

r = The annual rate of increase of the number of commercial vehicles (assumed to be 7.5 per cent unless more dependable local data is available)

n = The number of years between the last count and the year of completion of construction.

N = The design life of the pavement in years.

If adequate data is not available, it is recommended that average value of 7.5 per cent growth rate may be adopted for rural roads.

4.2.1.2.4 Design thickness

When the traffic projection is known and classified it into any one of the categories such as A,B,C, D etc., the design thickness of the pavement could be picked up from the **Figure 4-5** by making use of the appropriate curve. If the CBR values are available for the subgrade, sub-base and base, the total thickness as well as individual layer thickness can be obtained by repeated use of the curve.

4.2.1.3 Illustrated Examples on IRC Standard Axles Method :

Example-1 :

Design a flexible pavement with the following available data :

(a) CBR (with respect to standard proctor) :

Subgrade = 2 percent at 95% compaction

(b) Field data :

(i)	No. of commercial vehicles per day (in both directions),	A	=	100
(ii)	Annual growth rate of commercial traffic	r	=	8 percent
(iii)	Design life of the pavement,	x	=	10 years
(iv)	The terrain is plain; pavement with thin bituminous suurfacing. From Table-	F	=	2.25

Solution :

Projected traffic at the end of the design life of the pavement,

$$Ns = \frac{365 \times A[(1+r)^{x} - 1]}{r} \times F$$

$$Ns = \frac{365 \times 100[(1+\frac{8}{100})^{10} - 1]}{\frac{8}{100}} \times 2.25$$

$$= \frac{36500 \times 1.16}{0.08} \times 2.25$$

= 11,90,812 nos.

= 1.1908 Million (Cumulative number of standard axles, in design life)

Now, from the **Fig. 4-3, 4-4** and **Table 4.1** the layer thickness can be selected as given in **Figure 4-7 (A)**

Example-2

Design a flexible pavement with the following available data :

(a) CBR (with respect to standard proctor) :

Subgrade = 4 percent at 95% compaction

(b) Field data :

(i)	No. of commercial vehicles per day (in both directions),	A	=	300
(ii)	Annual growth rate of commercial traffic	r	=	10 percent
(iii)	Design life of the pavement,	x	=	12 years
(iv)	The terrain is hilly; pavement with thick bituminous suurfacing. From Table-	F	=	1.25

Solution :

Projected traffic at the end of the design life of the pavement,

$$Ns = \frac{365 \times A[(1+r)^X - 1]}{r} \times F$$

$$Ns = \frac{365 \times 300[(1+\frac{10}{100})^{12} - 1]}{\frac{10}{100}} \times 1.25$$

$$Ns = \frac{109500 \times 2.14}{0.10} \times 1.25$$

= 2929125 nos.

= 2.93 Million (Cumulative number of standard axles, in design life)

Now, from the **Fig. 4-3, 4-4** and **Table 4.1** the layer thickness can be selected as given in **Figure 4-7 (B)**

4.2.1.4 Illustrated Examples on IRC CBR Method

Example - 1 :

Design a flexible pavement with the following available data :

(a) CBR : (with respect to standard proctor)

(i)	Subgrade	=	2	Percent at 95% compaction
(ii)	Improved Subgrade	=	8	Percent at 98% compaction
(iii)	Sub-base	=	30	Percent at 100% compaction
(iv)	Base	=	70	Percent at 100% compaction

(b) Field data :

(i) No. of commercial vehicles as per last count (in both directions), P =100
(ii) Annual rate of traffic increase, r =6.5 percent
(iii) Expected period of completion of the road after last count, n =2 years

(C) Design life of the pavement, N=10 years.

Solution :

Projected traffic at the end of the design life of the pavement,

A = $P(1+r)^{n+N}$

= $100(1+6.5/100)^{2+10}$

= $100(1+0.065)^{12}$

= $100(1.065)^{12}$

= 100 (2.129)

= 213 numbers of commercial vehicles per day in both directions. In case of a Double-lane road this figure should be adopted.

For a Single-lane road, A = 2 x 213 = 426 numbers of commercial vehicles per day in both directions.

These refer to the Traffic Classification **(Table 4.3)** under category D which corresponds to the design of the same designation in curve D. Hence, using the curve D from **Figure 4-5**, the following depths above each layer may be scaled out :

Above subgrade (CBR : 2%)	=	60 cm
Above Improved subgrade (CBR : 8%)	=	28 cm
Above Sub-base (CBR : 30%)	=	13 cm
Above base (CBR : 70%)	=	7.5 cm

But, if we want to provide DBST or bituminous wearing coarse less than 5 cm thick the whole depth, required above sub-base should be covered by base course as because DBST or bituminous wearing course less than 5 cm thick does not provide significant structural strength to the pavement. Hence the total depth of construction could be kept 60 cm and the other layers may be adjusted as :

Above Improved subgrade (CBR : 8%)	=	30 cm
Above Sub-base (CBR : 30%)	=	15 cm
Above base (CBR : 70%)	=	DBST/2.5 cm bituminous wearing course.

The layer thickness have been worked out in **Figure 4-8 (A).**

Example-2 :

Design a flexible pavement with the following information and field data.

(a) CBR : (with respect to standard proctor)

(i)	Subgrade	=	4	Percent at 95% compaction
(ii)	Improved Subgrade	=	8	Percent at 98% compaction
(iii)	Sub-base	=	30	Percent at 100% compaction
(iv)	Base	=	70	Percent at 100% compaction

(b) Field data :

(i) No. of commercial vehicles as per last count (in both directions), P=200
(ii) Annual rate of traffic increase, r =7.5 per cent
(iii) Expected period of completion of the road after last count, n =3 years

(C) Expected design life of the pavement, N=10 years.

Solution :

The Projected number of commercial vehicles at the end of the design life of the road :

$$A = P\,(1+r)^{n+N}$$

$$= 200\,(1+ 7.5/100\,)^{3+10}$$

$$= 200\,(1+0.075)^{13}$$

$$= 200\,(1.075)^{13}$$

$$= 200\,(2.56)$$

= 512 numbers of commercial vehicles per day in both directions. In case of a Double-lane road this figure should be adopted.

For a Single-lane road, A = 2 x 512 = 1024 numbers of commercial vehicles per day in both directions.

This refers to the traffic classification under category E which corresponds to the design of the same designation i.e., curve E. So, using the curve E from **Figure 4-5** the following depths above each layer may be adopted :

Above Subgrade (CBR : 4%)	=	48 cm
Above Improved Subgrade (CBR : 8%)	=	31 cm
Above Sub-base (CBR : 30%)	=	14 cm
Above Base (CBR : 70%)	=	7.5 cm

But, if we want to provide DBST or bituminous wearing course less than 5 cm thick, the total depth required above sub-base should be covered by the base course as because DBST or bituminous wearing course less than 5 cm thick does not provide significant structural strength to the pavement.

Now, if we provide thinner bituminous wearing course (thinner than 5 cm), the total depth of construction and each layer thickness may be adjusted as under :

Above Subgrade (CBR : 4%)	=	50 cm
Above Improved Subgrade (CBR : 8%)	=	30 cm
Above Sub-base (CBR : 30%)	=	15 cm
Above Base (CBR : 70%)	=	4 cm bituminous wearing course

Each of the layer thickness is presented in **Figure 4-8 (B).**

Note : **In case of any bent, proper superelevation should be provided based upon proper design.**

In case of design of new pavement on a road where no traffic is there before construction, the initial traffic should be assumed on the basis of traffic count on an identical road in the vicinity.

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5.0 CHARACTERIZATION OF LOCALLY AVAILABLE MATERIALS IN BANGLADESH

5.1 Sub-grade

The upper layer of the Embankment whether in cut or fill is termed as sub-grade. The strength of the sub-grade is a basic factor in determining the thickness of pavement and is evaluated by means of CBR tests. The thickness of sub-grade normally should not be less than 300mm because the CBR value of the layer below it is always less than 3%. In some projects if it is ensured that CBR of the layer below sub-grade is more than 3%, then the thickness of sub-grade may be 250mm & 200mm for the CBR value of 4% & 5% respectively.

The thickness of pavement will vary due to CBR value of sub-grade. CBR value also vary due to change of compaction. In rural roads, 100% MDD Standard Proctor is specified for sub-grade material for CBR value of 8%. But if the same material is compacted to 94% MDD modified proctor, then CBR value will be more than 15%. In many of the LGED Projects, equipment are available for compaction of 95% MDD modified. This advantage should be taken for obtaining better compaction and bearing capacity for the same material.

According to the Road Note-31(Fourth edition), the minimum thickness of Improved Subgrade is recommended when the subgrade CBR value is 3&4. When the subgrade CBR is 2, thickness of Improved Subgrade is suggested to be 300 mm even for very low volume of traffic. So depending on the CBR value of subgrade the thickness of Improved Sub-grade layer may be chosen from **Table 5.1.**

TABLE 5.1

Subgrade CBR Value	Thickness of Improved Subgrade
2 or less	300 mm
3 & 4	200 mm

According to Mr. Hodge Kinson, Materials Expert of O " Sullivan & Graham who prepared the" Guide to the Design & Construction of Bitumen Surfaced Roads in Bangladesh - BRRL, Dhaka, **Table 5.2** may be used.

TABLE 5.2

Subgrade CBR (Soaked)		Thickness of Improved Subgrade
Greater than	but Less than	
5%	8%	200 mm
4%	5%	250 mm
3%	4%	300 mm
2%	3%	450 mm

Minimum thickness of subbase shall be 150 mm for FRB for traffic volume more than 0.3 million ESA for the design life as per Road Note-31.

Minimum thickness of Aggregate Base Course or WBM shall be 200 mm for FRB having traffic volume more than 0.7 million ESA for the design life as per Road Note - 31.

The surfacing may be DBST, 25 mm Bituminous Carpet or any other suitable type as per designers preference.

In some areas, the soil itself will give CBR value ranging from 10 to 25%. In some areas fine sand is used to get minimum CBR value of 8%. Mostly it is thought that local soil will not give 8% CBR and imported or selected fine sand is suggested. If 30% to 50% local soil is blended with fine sand, CBR value may be attained from 15% to 40% when compacted to 100% MDD standard. From numerous experiments & field use, it is established that when filler is added to fine sand, density as well as CBR increases. Under RDP-4, LGED in Faridpur, this item has been used in several numbers of Rural Roads and CBR achieved were more than 25%. While blending the soil, it should be tested in soil laboratory; PI value of the combined blended material shall be less than 10.

Where the local soil will not give CBR more than 8%, then to reduce the thickness of pavement, selected or improved sub-grade material is suggested. This improved sub-grade material for 8% CBR value may be sand, silty or clayey sand or combination of sand-soil whose PI shall be less than 10. To specify F.M. for sand to be used as Improved sub-grade is not required by any Standards either American or British.

It is recommended that as a general practice the design for new construction should be based on the strength of samples prepared at optimum moisture content and dry density corresponding to Proctor compaction (standard or modified) as specified in field and soaked in water for a period of four days prior to testing.

For field control, range of moisture should be specified i.e. field moisture should be + 1% to - 2% when Field density is tested.

In some project areas, the CBR value of the Sub-grade or Improved sub-grade material may be more than 20% to 30%; in that case no Sub-base is required.

A summary of the tests results of soils, sand, sand-soil and aggregate-sand-soil mixtures collected from many RHD Road Projects and LGED Rural Road Projects are shown in **Article 5.5** for information.

For Road projects under RHD, Daulatdia - Jhenaidah - Kushtia, Daudkandi - Feni & Dohazari -Cox's Bazar Road sections, mostly locally available soils and only in Faridpur - Kushtia section blended fine local sand & soil gave more than 20% CBR (value achieved 20 to 35%) at a compaction of 95% MDD modified while the minimum CBR requirement specified was 20%. It is possible to get higher CBR if compaction requirement is increased. The designer should consider CBR value of materials at 100% standard compaction and in projects where consultants are engaged it may be 103% Standard Proctor. In most of the districts heavy static Rollers or 3 Ton vibratory Rollers are available in LGED. It will be economical to use heavy compaction equipment, where available, to achieve higher strength using the same local materials.

The classification of soil in **Appendix-A** is enclosed in this manual for better understanding of Highway materials.

5.2 Sub-base

Sub-base materials comprise natural sand, gravel, brick metal, crushed stone or any other material like stabilized soil which remains stable under saturated conditions. For granular materials, most of the Standards recommend a wide range of gradings and specify that the materials passing 425 micron sieve when tested in accordance with AASHTO or British Standard should have Liquid Limit and Plasticity Index of not more than 25 and 6 respectively. These requirements should be enforced.

The sub-base material should have minimum CBR of 25% for Rural Road type B. For very low trafficked rural roads, the CBR requirements may be relaxed to 20%. The CBR samples should be prepared at the dry density (specified) and optimum moisture content and should be tested after soaking the test specimen in water for 4 days.

(1) Brick aggregates used as Sub-base:- Mostly the Brick aggregates are used in sub-base. It is seen from the test results of many Road projects that 33% Brick aggregates blended with sand & soil i.e. in the proportion of 1:1:1 will give at least 25% CBR value compacted either 100% MDD standard or 95% MDD modified. Many of the Standards ignore the gradation but gives stress on CBR value and Plasticity requirements. Blending of aggregate, sand & soil should be specified in the standard specifications in details.

(2) Natural sand used as sub-base:- There are many studies that coarse sand can be used as sub-base.

As this sand contains no filler materials and is non-plastic in nature, this layer when compacted, still remains loose. By adding filler material i.e. soil with PI value less that 10, i.e. PI of combined material less than 6, the material will give higher dry density resulting in higher CBR and will be a stable layer when compacted. Materials study conducted by many International Consultants in RHD and in LGED, suggest that sand can be used as sub-base. In Faridpur under RDP-4, LGED, sand and sand-soil have been used as sub-base and results are appeared to be satisfactory as the compaction in the field is controlled. In some areas of Bangladesh, coarse sand is available and it could be used as sub-base.

(3) Gravels used as Sub-base:- In some areas shingles are cheaper than brick aggregates. This may be used as sub-base. Only 33% of gravels will meet the CBR requirements of sub-base.

5.2.1 Sub-base layer thickness

According to Road Note 31, the thickness of sub-base layer was 100mm for Improved sub-grade or sub-grade having CBR of 8%. In the revised Road Note 31, the thickness varies from 125 mm to 150mm for traffic volume up to 1.5 million ESA which will not exceed normally for FRB Roads during the design life of pavement. Selection of the layer thickness is dependent on the judgement of the Designer. This layer may be laid as a single layer which is compactible by available rollers at site.

For projects located in different parts of Bangladesh, the available materials for sub-base will vary appreciably and testing in the laboratory should be conducted to use the material in a cost- effective manner.

As this sub-base material is granular in nature it is easily compactible. For most of the projects preferably the density requirements should be 100% of MDD Standard and in some special cases where heavy roller is not available, the compaction requirement may be 97% Standard proctor. Most of the consultants in LGED Projects, specify 100% Standard proctor which is under specified, because there will be excessive deformation along the wheel paths within 2/3 years of completion of the project and the thin asphalt layer cannot take the bending stress and ultimately cracks appear in asphalt layer along the wheel paths. To avoid early failure, both sub-base & base layers should be compacted at 103% standard proctor. The coarse aggregate size for the sub-base shall be less than 40mm. The grading requirements of combined sub-base materials is shown in **Table 5.3** below, but there is a wide range and it is more than satisfactory if 33% bricks or gravels are used to satisfy all the quality requirements of sub-base. The material containing 33% Brick chips will be within the grading envelope of the standard grading limits shown in the Table given below:

TABLE 5.3

B.S. Sieve size mm	Percentage by mass of total aggregate passing test sieve
50	100
37.5	80-100
20	60-100
5	30-100
1.18	17-75
0.30	9-50
0.075	5-25

The standard grading requirements for soil-aggregate, base & sub-base and quality requirements as per AASHTO is shown in **Appendix-B** for better clarification.

5.2.2 Quality Requirements of Sub-base material

Brick aggregates shall be made of overburnt bricks or brick bats and free from dust and other foreign materials. The LAA value for coarse aggregate either from bricks, gravels or stone shall not be more than 50% and AIV shall not be more than 40%. The water absorption shall be less than 18%. But the designer may specify any change in values if he thinks necessary for a particular project. The maximum compacted layer thickness shall be 200mm but it may be relaxed if the trial test results show that compaction of thicker layer is possible with available roller.

5.3 Base Course

The base is the main load spreading layer of the pavement and the life of the road is dependent on the strength of this layer. Bases are normally made from overburnt broken brick aggregates or crushed stone aggregates. For Rural roads, brick aggregates are used for economic reasons.

5.3.1 Water Bound Macadam (WBM)

Macadam was a Scottish engineer who introduced in the early nineteenth century the idea of constructing roads composed of small size of stones held together by means of a binding material. With the advent of bituminous materials, the term macadam was qualified by (1) “Water Bound” if stone materials are held by the addition of water and fillers (2) “Dry Bound” if aggregates are held together by mechanical interlock (3) “Wet Mix” of graded stones are premixed with water and compacted.

5.3.1.1 Materials for WBM

The most common and desirable material for use as aggregates in WBM is broken stone aggregates. The stones may consist of hard variety or soft variety. The harder varieties are preferred for upper layers, whereas, the softer ones can be used for lower layers. In Bangladesh, stone materials are hard to get and costly as well; over-burnt bricks are usually used to minimize cost of construction.

5.3.1.2 Size and grading requirements of coarse aggregate

It is a common practice so far to use single size stones/brick chips in WBM mainly because of the necessity to accept hand broken materials.

Gradation requirements of Aggregate for WBM is shown in **Table 5.4.**

TABLE 5.4

Grading requirements of Coarse Aggregates and Choking Aggregates (Screening materials) of WBM.

Sieve Size (mm)	% Passing by wt - Coarse Aggregate	% Passing by weight - Choking Aggregate
37.5	100	-
20	80-100	-
12	35-70	-
9.5	0-10	100
2.4	0-5	75-95
600 micron	-	25-50
150 micron	-	02-15

The fraction passing no 425 micron sieve shall have liquid limit not greater than 25 and P.I. not greater than 6. Choking aggregates may be khoa screenings or sand or any combination of these which will meet the grading requirements and should as much as possible be of same materials as that of coarse aggregate.

An approximate test of compaction is to place a piece of metal or an one inch stone on the WBM surface and run the roller over it. If no print is made on the surface or no embedment resutls, the compaction may be considered adequate.

5.3.1.3 Strength requirements of WBM materials

Aggregate Impact value should not be more than 32 and LAA value not more than 40.The screenings should be properly graded and some binding materials (filler) may be required.

Plasticity Index of binding material should be up to 6 for surface treated WBM and not more than 9 for WBM having no surface treatment.

5.3.1.4 Construction Procedure for WBM

The thickness of each layer should be such that the compacted thickness normally does not exceed 75mm. After spreading stone/brick aggregate and forming surface to the required grade and camber it is compacted by power roller to achieve proper interlocking of aggregates. After coarse aggregates are thoroughly keyed and set by rolling, the screenings are spread uniformly to fill the interstices. Dry rolling and brooming are carried out. The surface is then sprinkled copiously with water, swept and rolled. The binding material is then applied in two or more successive layers. After each application the surface is sprinkled with water and the slurry is allowed to fill the voids. Rolling shall continue till all voids are filled in. The road section is allowed to dry overnight and a layer of sand or soil about 6mm thick is spread on the surface lightly and sprinkled with water and rolled. The surface is allowed to dry and set and then opened to traffic.

5.3.2 Wet mix Macadam or Graded Aggregate Base Course

In Bangladesh gravels & stones are available and may be cheaper in certain areas compared to the cost of brick chips. While preparing the projects, the availability of local materials and their cost should be evaluated.

5.3.2.1 Quality requirements of Base Course - Wet mix Macadam or Graded Aggregate Base Course

Coarse aggregates shall have a percentage of wear by the Loss Angels Abrasion test of not more than 50 and 40 in case of sub-base & base materials respectively. For brick aggregates the water absorption should be not more than 20% and 15% in case of sub-base and base materials if it is tested by B.S. Aggregate Impact value for sub-base and base course material should not be more than 40 and 32 respectively. The fine aggregates passing 425 micron sieve should not have LL more than 25 and PI more than 6. A little amount of plasticity is desirable and an absolute non-plastic is undesirable (PI value up to 5 is called non-plastic).

5.3.2.2 Size and grading requirements of coarse aggregate

Gradation requirements of Aggregate for Wet mix Macadam is shown in **Table 5.5.**

TABLE 5.5

Gradation requirements for Wet mix Macadam/Graded Aggregates Base Course.

Sieve size (mm)	Percentage by Weight Passing the Sieve
1 in.(25mm)	100
3/8 (9.5mm)	50-85
No. 4 (4.75mm)	35-65
No. 10 (2.00 mm)	25-50
No. 40 (0.475 mm)	15-30
No. 200 (0.075mm)	5-15

5.3.2.3 Construction Procedure for Wet mix Macadam

The blending of coarse aggregates with sand & soil is very important; blended material should be mixed with water before transporting & laying to avoid segregation. The Base -course layer containing 150mm thickness should be laid at a time and should be compacted at a moisture content near the optimum moisture. As soon as rolling is completed, the field density should be checked and moisture content should be maintained before the required density is achieved and prime coat is applied over the completed Base - course. If the surface gets dry, watering should be done before priming.

5.4 Conclusion

So long WBM was used in road construction in Bangladesh. For the last few years, graded aggregate base course is introduced in many foreign aided road projects both in RHD & LGED. The graded aggregate road base is the improved form of WBM and is in extensive use in developed and developing countries. In WBM, 100% brick aggregate is used, whereas in graded aggregate base 60% brick chips blended with 40% sand-soil mixture gives better strength. By using 50 to 60% of brick chips, it is possible to get more than 60% CBR at 100% Standard Proctor and more than 80% CBR when compacted to 98% Modified proctor. This was made possible in Faridpur under RDP-4 Projects (**Ref. Page 34-35**). In many road sections of Faridpur, only 50% of brick chips were used in base coarse which was found performing well with 25mm surfacing or 20mm DBST. The required grading chart is given in **Table 5.5**; the designers may have other options as well. The proportion of brick chips, sand & soil may be 3:1:1 which will satisfy the grading if the samples collected at the time of Field density tests and compared with the grading chart. Blending of brick aggregates and rolling should be done under strict supervision.

As the material is granular in nature and easily compactable, the compaction requirement should be 103% Standard proctor in projects where consultants are engaged and heavy rollers specially vibratory rollers are available. This efforts will minimize the early deformation due to traffic movement and reduce the chance of appearing cracks in Bituminous surfacing and will make the pavement more durable to perform well during its design life under normal maintenance.

According to old Road Note-31, 150mm base thickness was required for 0.5 million ESA & 200mm for traffic volume more than 0.5 million ESA up to 2.5mm ESA. According to the new Rode Note - 31, base thickness varies from 150mm to 200mm up to traffic volume of 1.3 million ESA for the design life. So designer should not select the road base thickness less than 150mm arbitrarily.

5.5 Summary of the test results of Materials available for Sub-grade, Sub-base & Base Course in different projects of Bangladesh (RHD & LGED projects)

A. Improved Sub-grade

Sl. No.	Proportion		Compaction (Heavy)		% CBR (Soaked)	
	Sand	Soil	MDD/ Kg/m^3	OMC	at 100% MDD	at 95% MDD
1.		100	1675PI-12	17.80	16	10
2.	100 FM-.98	0	1703	15.71	48	21
3.	100 FM-2.53	0	1960	11.00	34	28
4.	100 FM-0.53	0	1980	12	34	26
5. a)	100FM-0.60	0	1900	14.23	20	0
b)	0 FM-0.60	100	1760	12.50	25	20
c)	40 FM-0.60	60	2050	11.00	22	17
6. a)	100FM-0.75	0	1840	10.00	22	16
b)	0 FM-0.75	100	1790	13.30	18	13
c)	50 FM-0.75	50	2070	11.00	24	-
7.	Sylhet Sand	-	1770	13.50	35	25
8.	-	Lalmai Soil	1940	10.50	38	27
9.	-	Chowdha -gram Soil	2060	10.10	36	30
10.	-	Dohazari	2010	10.54	34	28

B. Sub-base- Minimum CBR requirement is 25%

Sl. No.	Proportion			Compaction (Modified)			% CBR (Soaked) at 95% MDD
	Agg.	Sand	Soil	MDD- Kg/m^3	OMC	at 100% MDD	
1.	1	1	1	1815	14	-	61
2.	1	1	2	1825	15	-	43 (93% Compaction
3.	1	1	1	1790	15.5	-	60 (98% Compaction)
4.	40	30	30	1780	16	-	63 (98% Compaction)
5.	40	30	30	1770	15.7	-	65 (98% Compaction)
6.	33	27	40	1867	-	52	45
7.	35	25	40	1837	-	71	55
8.	35	35	30	1910	-	82	65
9.	35	25	40	1838	-	64	48
10.	40	30	30	1825	-	72	52
11.	30	20	50	1850	-	56	40
12.	30	30	40	1802	-	90	32

C. Aggregate Base Course - Minimum CBR Requirement is 60%

Sl. No.	Proportion			Compaction Modified		% CBR-Soaked		Remarks
	Agg.	Sand	Soil	MDD Kg/m^3	OMC	at 100% MDD	at 95% MDD	
1.	40	30	30	1.778	13.50	125	60	PI-6
2.	45	30	25	1.773	15.80	136	70	PI-6
3.	80	20	-	1.759	17.00	130	100	
4.	60	40	-	1760	16	128	72	
5.	50	30	20	18.54	15	125 (97% Compact)	-	
6.	50	30	20	17.48	18	88	-	Standard 100% Compaction)
7.	30	40	30	19.67	11	78	-	(98% Compaction)

D. SUMMARY OF INFORMATION ON ASS PAVEMENT IN RDP-4 DISTRICT.

DISTRICT : FARIDPUR

Sl. Nos.	Scheme Name	Scheme Information From HBB to ASS/Earth road improved	Avg. No.of bus & trucks daily Status	Design information					Field Attainment					PI Value of soil Sub-base(SB) Base(B)
				Sub-grade Avg. value	Improved Sub-grade Avg. value	Sub-base Avg. value	Base Mix. Propo. Avg. value	Surfacing	Sub-grade Avg. value	Improved Sub-grade Avg. value	Sub-base Avg. value	Base Mix. Propo. Avg. value	Surfacing	
1.	RCTP-III Section 3-1 400.00m	Trial Scheme Scheme dura-tion 1992-93 HBB improve ment	Traffic=68 Completed	CBR=4% Thickness = 125mm	CBR = 7% M.Comp. = 90% Thickness = 100mm	SS=1:1 CBR = 20% M.Comp=93% Thickness = 275mm	ASS=40:30:30 CBR=60% M.Comp=98% Thickness =200mm	Surface dressing 15mm	CBR=11.9% M.Comp=90% Thickness =125mm Test no. =11 (CBR) Test no.=24 (Com)	CBR=25% M.Comp=90% Thickness =100mm Test no.=11 (CBR) Test no.=24 (Com)	SS=1:1 CBR=27.9% Comp.=93% Thick=275mm Test no.=11(CBR) Test no.=24(Com)	ASS=40:30:30 CBR=60.7% Comp.=98% Thick=200mm Test no.=11(CBR) Test no.=24(Com)	Surface dressing 15mm	SB=9 B=6
2.	Section 3-2 200.00m	Trial Scheme Scheme dura-tion 1992-93 HBB improve ment	Traffic=68 Completed	CBR=4% Thickness = 125mm	CBR = 7% M.Comp. = 90% Thickness = 100mm	SS=1:1 CBR = 20% M.Comp=93% Thickness = 200mm	ASS=40:30:30 CBR=60% M.Comp=98% Thickness =150mm	Bitumin-ous pave-ment 40+ 7 mm	CBR=11.3% M.Comp=90% Thickness =125mm Test no.=6 (CBR) Test no.=12(Com)	CBR=27% M.Comp=90% Thickness =100mm Test no.=6 (CBR) Test no.=12(Com)	SS=1:1 CBR=29.5% M.Comp.=93% Thick=200mm Test no.=6(CBR) Test no.=12(Com)	ASS=40:30:30 CBR=64.6% M.Comp.=98% Thick=150mm Test no.=6(CBR) Test no.=12(Com)	Bitumi-nous pave-ment 40+ 7mm.	SB=9 B=6
3.	ENVIRON-MENTAL TRIAL ROAD-1 Section 1-1 1600 m	Trial Scheme Scheme dura tion 1993-94 HBB improve ment	Traffic=68 Completed	CBR=4% min. Thickness= 150m	CBR = 7% M.Comp. = 90% Thickness = 150mm	Sand=150mm CBR=20% M.Comp=96%	WBM=75mm ASS=40:30:30 CBR=65% Thickness =150mm	Single Otta 20 mm	M.Comp=90% Test no.=41(CBR) Test no. =96(Com)	CBR=11.3% M.Comp=90% Thickness =150mm Test no.=41(CBR) Test no.=96(Com)	Sand sub-base =150mm CBR=28.7% M.Comp.=93% Test no.=41(CBR) Test no.=96(Com)	ASS=40:30:30 CBR=62.7% M.Comp.=98% Thick=75+ 150mm Test no.= 41(CBR) Test no.= 96(Com)	Single Otta 20 mm	SB= ---- B=6
4.	Section 1-2 200.00m	Trial Scheme Scheme dura tion 1993-94 HBB improve ment	Traffic=68 Completed	CBR=4% min. Thickness= 150m	CBR = 7% M.Comp. = 90% Thickness = 150mm	Sand=150mm CBR=20% M.Comp=96%	WBM=75mm ASS=40:30:30 CBR=65% Thickness =150mm	Double of with 30% crushed stone (35mm)	M.Comp=90% Test no.=6(CBR) Test no. =12(Com)	CBR=12.7% M.Comp=90% Thickness =150mm Test no.=6(CBR) Test no.=12(Com)	Sand sub-base =150mm CBR=28.4% M.Comp.=93% Test no.=6(CBR) Test no.=12(Com)	ASS=40:30:30 CBR=60.7% M.Comp.=98% Thick=75+ 150mm Test no.=6(CBR) Test no.=12(Com)	Double Otta 35 mm	SB= ---- B=6

Sl. Nos.	Scheme Name	Scheme Information From HBB to ASS/Earth road improved	Avg. No.of bus & trucks daily Status	Design information					Field Attainment					PI Value of soil Sub-base(SB) Base(B)
				Sub-grade Avg. value	Improved Sub-grade Avg. value	Sub-base Avg. value	Base Mix. Propo. Avg. value	Surfacing	Sub-grade Avg. value	Improved Sub-grade Avg. value	Sub-base Avg. value	Base Mix. Propo. Avg. value	Surfacing	
5.	ETR - 11-1 400.00m	Trial Scheme Scheme dura tion 1993-94 HBB improve ment	Traffic=68 Completed	Existing sand =100mm CBR=8% M.Comp=88%	Sand=75mm CBR = 14% M.Comp. = 90%	ASS=1:1:2 CBR=25% M.Comp=93% Thick=75mm	WBM=75mm ASS=40:30:30 Thick=100mm CBR=60% M.Comp=98%	Asphalt concrete 25mm dense	Existing sand =100mm M.Comp=88% Test no.= 11(CBR) Test no.= 24(Com)	Sand=75mm CBR=14.7% M.Comp =90% Test no.= 11(CBR) Test no.= 24(Com)	ASS=1:1:2 Thick=75mm CBR=39.4% M.Comp. =93% Test no.= 11(CBR) Test no.= 24(Com)	WBM=75mm ASS=40:30:30 Thick=100mm CBR=63.7% M.Comp.=98% Test no.= 11(CBR) Test no.= 24(Com)	Asphalt concrete 25mm dense	SB= 9 B=6
6.	ETR - 11.2 400.00m	Trial Scheme Scheme dura tion 1993-94 HBB improve ment	Traffic=68 Completed	Existing sand =100mm CBR=8% M.Comp=88%	Sand=75mm CBR = 14% M.Comp. = 90%	ASS=1:1:2 CBR=25% M.Comp=93% Thick=75mm	WBM=75mm ASS=40:30:30 Thick=100mm CBR=60% M.Comp=98%	DBST =25mm	Existing sand =100mm M.Comp=88% Test no.=11(CBR) Test no.=24(Com)	Sand=75mm CBR=18% M.Comp =90% Test no.= 11(CBR) Test no.= 24(Com)	ASS=1:1:2 Thick=75mm CBR=41% M.Comp.=93% Test no.= 11(CBR) Test no.= 24(Com)	WBM=75mm ASS=40:30:30 Thick=100mm CBR=58% M.Comp.=98% Test no.= 11(CBR) Test no.= 24(Com)	DBST = 25mm	SB= 9 B=6
7.	63.00m BHUB-ANESHWA BRIDGE APPROACH 120.00M	Scheme dura tion 1993-94 HBB improve ment	Traffic=10 Completed	CBR= 4% min.	Sand= 125mm CBR = 7% M.Comp. = 90%	Sand sub-base=225 CBR=20% M.Comp=93%	ASS=1:1:1 CBR=60% M.Comp=95% Thick=200mm	Bitumi-nous carpeting 25mm dense	----	Sand= 125mm CBR=17.2% M.Comp =90% Test no.= 4(CBR) Test no.= 9(Com)	Sand sub-base CBR-42.2% M.Comp=93% Test no.= 4(CBR) Test no. =9(Com)	ASS=1:1:1 CBR=61% M.Comp=95% Thick= 200mm Test no. =4(CBR) Test no. =9(Com)	Bitumi-nous carpeting 25mm dense	SB= ---- B=6

6.0 BITUMINOUS SURFACING

The most important and critical layer of the pavement is the bituminous surfacing. The performance of Sub-base & Base is dependant on the durability of this layer. So the selection of type of surfacing and materials for this layer is to be done very carefully. Following types of surfacing are being used in LGED projects specially RDP-7, RDP-16 & RDP-18 and their performance are quite satisfactory where strict supervision were done by engaging trained staff during execution .

(1) Densely graded Bituminous Carpeting (BC)
(2) Double Bituminous Surface Treatment (DBST)
(3) OTTA surfacing
(4) Seal Coat (7mm/12mm)

The volume of traffic in rural roads in our country is relatively low. In most of the cases it is not justified to use more that 25mm B.C. or 20mm. DBST. According to the revised Road Note 31, DBST over granular road base can be used for any volume of traffic. The cost of DBST is almost same to that of 25mm B.C. Both the surfacings will have durability of more than 8-10 years, but surface texture and riding quality of B.C. is excellent. However considering the long-term performance and maintenance, the use of DBST should get preference. In many of the RHD Roads, DBST's performance is found to be excellent. Both the types are being used in LGED Road Projects.

Normally open graded materials and hand mixing are being used in premixed B.C. This open textured surfacing requires Seal coat. But now dense graded materials are used in some of the LGED Projects without Seal coat and there is a substantial cost savings along with better quality and durable job.

6.1 Coarse Aggregates

Stones, boulders and gravels are found in 4 regions in Bangladesh and all of those materials can be used but must be tested and should meet the quality requirements as shown in Table 8.1 of Road Note 31. Comparatively softer aggregates will give more stability than the harder rock like Pakur Stone, but a slightly higher bitumen will be necessary. In case of B.C. the performance of Sylhet stones will be better than Pakur stone. The L.A.A. Value should be less than 40%. The stone chips should not be broken under roller while compacting the hot mix. As the item of work is very costly, the selection of aggregates should be done very carefully. Only broken stone should be used instead of shingles.

6.2 Fine Aggregates

Fine aggregates may be natural sand, crushed stone dust and filler i.e. stone dust. PI value of the fine aggregates shall be less than 4 and Sand equivalent value should be more than 50. Sand equivalent value is determined by performing Sand equivalent test to find plastic fines in aggregates & soils. This test is intended to serve as a rapid field test to show the relative proportions of fine dust or clay, claylike materials in soils or graded aggregates.

6.3 Preparation of Marshall Mix Design

The asphalt mix shall be tested as per Marshall method in the laboratory so as to meet the Marshall test criteria and to determine the optimum bitumen content. As the surfacing should be flexible, to make it durable, the combined grading should be on the finer side and bitumen content should be as high as practicable to avoid any bleeding.

6.4 Use of Aggregates in DBST

The aggregates for DBST should also meet the quality requirements same as B.C. In selecting the nominal size of chippings for DBST, for the first layer it should be 14mm and for the second layer it should be 6mm. The chippings of the second layer may be crushed or uncrushed. These sizes are suggested because the volume of traffic is low in rural roads.

6.5 Prime and Tack Coat

There is a difference in materials to be used in prime & tack coat and their purposes are different. The quantity of bituminous material is also too much variable. It is clearly stated in Road Note-31. The prime coat material is Medium Cut back bitumen or bitumen is blended with kerosene or diesel; Tack coat material is Rapid Curing bitumen of 80/100 or 60/70 grade bitumen. Prime coat is used over the unbound layer of Base, Sub-base or Subgrade materials. Prime Coat materials will be absorbed and the surface will be tight and water proof. Surfacing will be allowed after at least 48 hours or more. The quantity of prime coat material shall be 1.00 - 1.5 litre per m^2 depending on the surface condition.

The tack coat will be used on the old bituminous bound layer to get better bond between the new & old surface. The quantity of tack coat material shall be 0.25 to 0.4 litre per m^2 & it shall be sprayed before the surfacing starts. The areas which shall be tack coated, should be covered in the same day.

7.0 ROAD DRAINAGE

7.1 Introduction

One of the worst enemies of roads and highways is the water. It has been identified since roads were first built that their stability can only be maintained if the surface and foundation remain in a relatively dry condition. Water reaches the roadway mainly due to accumulation of rain water on the road surface, flow of surface water from the adjoining land, occasionally due to the overflowing of water courses and percolation through soil. Water brings about the deterioration, failure and destruction of the roads and highways by :

(i) Softening the road surface of an unpaved road when constructed of soil, or sandy-clay or gravel or water bound macadam.

(ii) Washing out unprotected areas of the top surface, erosion of shoulders, side slopes forming gullies, erosion of side drains, etc.

(iii) Softening the subgrade soil and decreasing its bearing capacity.

(iv) Penetrating into the paved bituminous surface through cracks reducing its stability.

(v) Softening the ground above and below the road where it passes along the high ground, causing land slides or slips.

7.2 Requirement of Road Drainage System

It has become a universally accepted fact that the drainage of the road surface, road pavement layers and subgrade is one of the most important aspects of road construction. Nobody can deny that a well drained road will have a far longer life than a poorly drained road. If an additional expenditure is incurred on drainage, it shall always be recovered in terms of longer life and reduced maintenance cost of the roads. Case studies reveal that many road failures arise from poor drainage facilities. However, a good drainage system for highways would require :

(i) Side drains wide and deep enough to carry away all water that accumulates therein to some drainage system. Water level in these drains should remain at all times below the subgrade level.

(ii) Intake and outlet of the drains shall be wide and deep enough to carry the water that is brought to them.

(iii) Adequate crown along the central line of the road or sufficient cross slope to drain off water as quickly as possible that falls on the road surface without allowing it enough time to percolate.

(iv) All springs and underground sources of water to be tapped and drained by sub-surface drains.

(v) Construction of intercepting drains parallel to the road but outside the road limit to intercept water before it reaches the road in such areas where the topography forces the water to flow towards the roadway itself.

(vi) Adequately designed drainage structures to drain off water immediately without overflowing.

Drainage, in broad terms, can be divided into two categories, namely, surface drainage and sub-surface drainage. Surface drainage can be defined as the measure taken to control the flow of surface water while the subsurface drainage is taken to control ground water in its various forms.

7.3 Surface Drainage

In case of new construction of a highway, surface drainage should be thought of and planned at its survey stage. An ideal alignment for a highway from drainage point of view is along the divides ridge line. All the streams would then flow away from the highway and the surface drainage problem would be reduced to tackling the water that falls within the roadway boundary only.

The elevation of road surface should always be so designed that it can shed water as quickly as possible during rain. Standing water is a serious hazard to traffic in one hand and would eventually penetrate through the surfacing into pavement and subgrade layers reducing their stability on the other.

The prerequisite and precondition of a good surface drainage is to provide an impermeable surfacing of a road. Even though the best surfacing materials are used in surfacing it will eventually become permeable unless a highest standard maintenance is carried out. In the truest sense of the term, it would be totally unjustified to assess that water will never penetrate the pavement layers and reach to the subgrade during the life of the road.

In spite of an impermeable surfacing is provided, due care must be taken to lay wearing surface of the pavement to the correct cambers or cross-slopes so that no low spot is left anywhere on the surface. Experience shows that wherever water stands on a road surface, the sign of deterioration like cracks, potholes, settlement, etc. starts from that particular place and expands progressively.

Depending on the topography and land use in the alignment route, the disposal of surface water can broadly be divided into three categories :

(a) Drainage in rural area,
(b) Drainage in urban area,
(c) Drainage in hilly terrain.

7.3.1 Drainage in Rural Area

Most of the roads in rural plain area are in embankment or on ground line. When the road is in the embankment, it is a common practice to allow water on the road surface to flow across the shoulder and down to the side slopes. Open side drains are generally provided along the highways to collect and discharge water that comes down the slope in order to prevent the surface water from flowing over the cultivated or otherwise improved land. Flat bottom as well as V-shaped side drains may be used but preference may be given to V-shaped one, because of its higher capacity with increased depth. In either case, side slopes are made as flat as possible consistent with drainage requirements and limiting widths of right of ways. Deep and narrow side drains are to be avoided wherever possible because of the increased hazards presented by such construction. Where their use cannot be avoided, adequate provision must be made to safeguard traffic through the use of guard rails and similar devices.

Grades used in open drains may also be roughly the same as those are used on the highway centre line. On the other hand, flat roadway grades and steeper drain grades in the same location may be used as and when required.

In a very flat country, drain grade as low as 0.1 or 0.2 per cent may be used while in rolling and mountainous terrain the maximum allowable grade may be used. A section of road along with side drain to be used in rural plain area is shown in **Figure 7-1(A).**

7.3.2 Drainage in Urban Area

In urban areas, surface drainage is a bit difficult and complicated matter. Drainage problems in the major cities like Dhaka, Khulna and others are the burning issues to the city authorities. The drainage problems in the urban areas are mostly due to the reasons as under:

(i) Absence or unusable water courses within the urban area.

(ii) Impervious nature of the area which results in the generation of very high surface run-off.

(iii) Non-availability or excessive cost of land for side drains.

(iv) Undesirability of open drains due to risk involved to road users, unsightly appearance, unhygienic dumping of garbage and menace of flies, mosquitoes and other insects.

Sometimes in relatively smaller urban areas open side drains are provided along the edge of the street. But in most cases, water from the road surface is taken to a system of underground drains or pipes known as storm drain or storm sewers and carried in them over a considerable distance and released again as surface run-off. This collection and transmissions make the problem still difficult because the entire water must be collected with no ponding, thereby eliminating natural storage and through increased velocity, the peak run-off condition is reached quickly. Sections of drains normally being used is shown in **Figure 7-1(B).**

7.3.2.1 Curb and Gutters

Curbs and gutters are sometimes used at the outer edge of the traveled way in the urban areas to prevent the water that flows from the crowned surface from spilling over and eroding the shoulder and side slopes. They also help to provide a measure of protection to pedestrians as well as facilitates drainage. Only the curb or the combination of curb and gutter may be used for the purpose. At certain intervals, water that collects against the curb or on the gutter is made to flow through the gratings into the gullies and is drained off through an outlet pipe to a storm sewer which takes the water away to a natural water disposal point. The arrangement of the disposal of water of the type is shown in **Figure 7-2.**

7.3.3 Drainage in Hilly Areas

Road network in the eastern and northern region of the country passes through hilly terrain in many places. For the proper maintenance of this road network in hilly areas an effective drainage system is very important. It is a normal practice to drain off surface water in hilly roads by side drains or by catch water drains. Unlike the case of plains where side drains are provided on both sides of the road, for hill roads, side drains are normally provided only one side of the road. However, drains on both sides of the roads can only be provided when it runs through cutting. Moreover, if any drain is located by the side of the road for surface water drainage, it will reduce the effective width of the roadway. As such, it may be considered essential that the drain should be of such a shape so that it could function both as a drain and also as a part of road surface in case of emergency. Thus, if a vehicle is forced to move to the extreme edge of the road, even inside the drain to avoid an accident, it should be able to come out easily. In order to match this, side drains are made to be either angular or parabolic or fixed with curb as in **Figure 7-3.**

The minimum depth of side drain should not be less than 150mm and the section should be suitably enlarged to meet the actual requirements of the discharge. Wherever the side drain passes on the soft formations, the drains should be lined appropriately.

When the road passes through the steep side of a high ground, the water from the upper slope would rush on the road and wash it away. This can be prevented by interrupting the water at some distance away from the road edge by means of what are commonly known as catch drains. Thus where slips are likely to occur and where large quantities of the surface flow are likely to generate on to the road, catch water drains should be provided. This catch water drain is usually kept on the same side of the side drain and generally laid along the contours up the hill to intercept the run-off from the hill slope and divert it to the nearest cross drainage work such as a culvert or to a natural stream. A careful watch should be kept on the condition of these drains in soft strata so that no damage is done to the hillside by percolation of water from these drains. The lower side and bottom of a catch water drain should be lined with rough stone or brick if the soil is easily erodable. These drains should not be located closer than 4.5m from the edge of the road to avoid land-slide due to seepage of water through down the slope. The normal size of a catch water drain is 1m by 1m.

7.4 Sub-Surface Drainage :

Stability of a road pavement depends upon the strength of its subgrade. On the other hand, strength of subgrade also depends on its density and moisture content. For the pavement thickness design, the subgrade strength is assumed at certain moisture content and if this assumed limit of moisture content is exceeded, the designed strength of subgrade is bound to reduce with the decrease of subgrade stability causing its settlement. As such, if the subgrade is kept at or below of its assumed moisture content, the stability of the road structure could be maintained. With this end in view, it is required to minimise the sub-surface flow by providing with adequate sub-surface drainage arrangements.

7.4.1 Causes of Moisture Variation in Subgrade

Moisture content of subgrade soil changes due to a number of causes. Some of them are as under:

(i) Seepage of water into the subgrade from adjacent higher ground.
(ii) Rise or fall in the level of water-table.
(iii) Percolation of water into the subgrade through cracks or porosity of the road surface.
(iv) Transfer of moisture by capillary action from lower layer of soil.
(v) Transfer of moisture from shoulder to the pavement edges.
(vi) Transfer of water vapour through soil.

The changes of subgrade moisture content in various ways is shown in **Figure 7-4.**

7.4.2 Sub-surface Drainage System :

The most common practice adopted for sub-surface drainage for the pavement and subgrade in rural plain area is to provide subgrade drains with granular materials at certain intervals on both sides of the road at staggered position. Typical arrangements of this subgrade drain is shown in **Figure 7-5.** The drainage material must have a compromise in granular size so that it can allow the passage of water but not too large to allow fines to be washed away from the pavement. A blend of fine gravel and coarse sand could be a very good combination for the purpose. A fabric separator between the drain and pavement/shoulder materials may be used to prevent fine particles from washing out of the drain. It must be ensured that subsequent work on the embankment side slopes does not contribute in the sealing of the drainage layer.

Sometimes ground water may be found over an impervious strata in an inclined layer in the hilly region which may seep into the subgrade from the upper side of the road. This running water through the previous soil layer may be checked by providing a drainage trench on the hill side to intercept water and drain it to a suitable disposal point. The arrangements of an open intercepting drain are shown in **Figure 7-6(A*)* *and* Figure 7-6(B*)*** which are self explanatory.

Wherever the seepage zone exists within 1m of the surface of the road, a perforated drainage pipe is installed just in the impermeable strata underlying the seepage zone as illustrated in **Figure 7-7(A)** . In case the seepage zone is very wide or the impermeable strata underneath is very deep, it is not economical to put the trench upto the impervious strata and intercept all the seepage water. A drain is normally provided in order to keep the water table about 1.2m below the formation level of the road. This arrangement of partial interception of a deep seepage zone is shown in **Figure 7-7(B).**

Most of the rural roads in the country passing through plains are constructed with low embankment. Free-board of these roads during rainy season stands to a minimum with standing water on both sides. Consequently the water table rises and comes very close to the pavement. As a result the stability of the subgrade is reduced and causes a major threat to the pavement as a whole. In order to overcome this problem, a drainage system as shown in **Figure 7-8** may be installed to lower down the water table. In case a perched water table exists, vertical drains are provided to transfer water to the main water table.

Note : See also Article-5 "Drainage and Road shoulders" extracted from Road Note-31 and given in Appendix-D of this Manual.

8.0 CONCRETE PAVEMENT

Concrete Pavement is not generally used in Bangladesh though in early sixties it was used in many National & Regional Highways. The use of concrete pavement may be costlier for a short design life, but its use should be mandatory for rural roads located in areas where the road is submerged or inundated by flush flood or unusual flood water. Some of the countries are now using concrete pavement; Chile is one of the country where concrete pavement is being used successfully. This type of pavement is also called as Rigid pavement.

8.1 Selection of Concrete Pavement

In Bangladesh flexible pavement is widely used. But the designer must not choose only one type neglecting the other without comparing their economic benefits.

The selection of pavement type shall be based on the governing factors such as (1) Traffic, (2) Soil characteristics, (3) Weather, (4) Performance of similar pavement in the Area, (5) Economic cost comparisons etc.

(a) **Traffic :** For heavy traffic and overloaded trucks, where the damaging factors are high, the Rigid Pavement is preferable to avoid early damage of pavement.

(b) **Soil characteristics :** Rigid pavement can be built on any soil condition even on sub-grade strength of CBR 2 where as flexible pavement needs strong foundation soil. The load carrying capacity of concrete pavement is mainly due to the rigidity and high modulus of elasticity of the slab itself. Dr. Westergaard, pioneer of rigid pavement design, considers the rigid pavement slab as a thin plate resting on soil sub-grade, which is assumed as a dense liquid, since reasonably uniform support rather than strong support is the most important function of the sub-grade for concrete pavement. Concrete pavement needs very little maintenance, if it is well designed and constructed with the modern technique.

(c) **Weather :** Weather in Bangladesh where the unavoidable flooding is a normal phenomenon; there is no other choice except to use rigid pavement in many areas of the country.

(d) **Performance of similar pavement in the Area:** The concrete pavement which was constructed in many parts of the country during the period of 1950 -1960, was found to be workable upto 1986. The performance was more than satisfactory if we consider the thickness which was very less, not properly designed and the work was done almost without engineering supervision and without any quality control system. Now the consultants are engaged for proper design & supervision of the site and the concerned department is quality conscious.

(e) **Economics of cost comparision:** For design life of more than 10 years, the Rigid pavement may be economical for medium & high volume of traffic. From a study of Highway project under RHD, it was observed that initial cost is not higher in case of Rigid Pavement. What RHD is spending for construction of one km of flexible pavement for a design life of 10 years, with that investment one km of Rigid pavement can be built for a design life of 25 years. For rural Roads like FRB, a study can be made to find out the actual cost. One Road in Rupganj under Narayanganj District was constructed with Rigid pavement in 1994. The cost and performance can be evaluated and compared with other flexible pavement roads in those areas.

8.2. Design of Concrete Pavement

8.2.1 Sub-grade and Sub-base

The load bearing capacity of the sub-grade is not important in the design of concrete roads, but uniformity in sub-grade is essential. In the design of concrete roads, three qualities of sub-grade are considered and shown in **Table 8.1**. For weak sub-grade i.e. CBR value of 2 or less, a sub-base having 150mm minimum thickness is required. When sub-grade with sand or sand-soil having CBR 2 to 14%, a sub-base thickness of 80mm is required. When the CBR value of sub-grade is 15% or greater no sub-base is required. But in worst soil condition, a layer of 80mm thick sub-base for CBR value of 15% is recommended to protect the pavement from mud pumping.

TABLE 8.1

Classification of subgrades for concrete roads and minimum thickness of sub-base required

Type of subgrade	Definition	Minimum thickness of sub-base required
Weak	All subgrades of CBR value 2 percent or less	150 mm
Normal	Subgrade other than those defined by the other categories	80 mm
Very stable	All subgrades of CBR value 15 percent or more. This category includes undisturbed foundations of old roads	0

For high volume of traffic i.e. Regional or National highways and also for rural roads, it is necessary to provide a sub-base layer treated with cement (1:4:8 proportion) over the sub-grade to prevent mud pumping. This sub-base will help to control the volume changes for severe conditions in sub-grade material.

To take care of the poor quality sub-grade, a cement treated sub-grade layer of 200 mm thick, shall be provided. This has become a practice to the developed countries. The principal benefits of cement sub-grade are as follows :

i) A uniform & strong, non-consolidating support is provided for the pavement.
ii) Firm support for side forms contributes to the construction of smoother pavement.
iii) Provides an all weather working platform which will help to work throughout the year.

8.2.2 Concrete Slab

The concrete slab may be unreinforced, reinforced, continuously reinforced, or pre-stressed concrete. A simplified design for unreinforced and reinforced slab is taken for this manual. Road Note 29 and latest review made by Mr. Croney and Croney based on the full scale experimental length on Public Roads in Great Britain over the last 60 years are the basis for the design considerations.

Design thickness charts for concrete Road are shown in **Figure 8-1 and Figure 8-2**.

It is clear from the charts that thickness of concrete road for cumulative traffic of 2 million ESA for a design life of 20 years is 100mm to 125 mm and the variation of reinforced and unreinforced slab is minimal.

For concrete slab, the compressive strength of concrete should be minimum 245 N/mm^2 (3500 psi). This strength can easily be achieved with the cement presently being used for 210 N/mm^2 (3000 psi) concrete, because the slump of concrete for pavement may be 10mm to 25mm. It should be ensured that quality of cement is acceptable. Maximum aggregate size may be 40mm and minimum cement content be 350 kg per cubic meter.

For formulating design of Concrete Pavements, the traffic, the design life, the subgrade, the sub-base and the concrete slab are considered in this section of the manual. The traffic for the design life is discussed in a separate chapter of this manual.

An Example of thickness design of Rigid Pavement is given below:

For determination of thickness, design charts are shown in **Figure 8-1 and Figure 8-2** from Road Note-29 and from a latest publication of Mr. Croney & Croney relating to design of concrete pavement.

Pavement thickness for Design life of 20 years:-

Cumulative Equivalent Standard Axle = 2 Million

A. Over weak sub-grade

1. Sub-grade CRB = 2% , thickness = 200 mm
2. Sub-base thickness requirement = 150 mm (**Table 8.1**)
3. Concrete slab thickness requirement = 180 mm - (both reinfroced & unreinforced**: Fig. 8-1**)
   = 150 mm - (unreinforced **- Fig. 8-2)**
   = 120 mm - (reinforced **- Fig. 8-2)**

B. Stable Sub-grade

1. Sand-soil Sub-grade - CBR 15%, thickness = 200 mm
2. Sub-base thickness requirement = Nil (**Table 8.1**)
   (80mm Sub-base is recommended to prevent mudpumping & better performance in the long run)
3. Concrete slab thickness requirement = 180 mm - (unreinfroced & reinforced **: Fig. 8-1**)
   = 150 mm - (unreinforced - **Fig. 8-2)**
   = 120 mm - (reinforced - **Fig. 8-2)**

8.2.3 Reinforcement for Concrete Slab

For reinforced concrete, minimum weight of reinforcement required in relation to the cumulative number of standard axles to be carried during design life is shown in **Figure 8-3**.

According to Road Note 29, minimum reinforcement of concrete slab in relation to the spacing of joints has been provided in **Figure 8-4**.

If the designer intends to make the Rigid Pavement reinforced, he may use the design chart or calculate the reinforcement as shown below:-

a) Reinfrocement, when provided in concrete pavements, is intended for holding the fractured faces at the cracks tightly closed together, so as to prevent deterioration of the cracks and to maintain aggregate interlock thereat for load transfer. It does not increase the flexural strength of unbroken slab when used in quantities which are considered economical. Where the slabs are provided adequately with joints to control cracking, such reinforcement has no function.

b) Reinforcement in concrete slabs is designed to counteract the tensile stresses caused by shrinkage and contraction due to temperature or moisture changes. The maximum tension in the steel across the crack equals the force required to overcome friction between the pavement and its foundation,

from the crack to the nearest joint or free edge. This force is the greatest when the crack occurs at the middle of the slab. Reinforcement is designed for the critical location. However, for practical reasons, reinforcement is kept uniform throughout the length, for short slabs.

The amount of longitudinal and transverse steel required per metre width or length of slab is computed by the following formula :

$$A = \frac{LFW}{2S}$$

in which

A = area of steel in cm^2 required per metre width or length of slab,

L = distance in m between free transverse joints (for longitudinal steel) or free longitudinal joints (for transverse steel),

F = coefficient of friction between pavement and subgrade (usually taken as 1.5),

W = weight of slab in kg/m^2, and

S = allowable working stress in steel in kg/cm^2 (usually taken as 50 to 60 per cent of the minimum yield stress of steel).

c) Since reinforcement in the concrete slabs is not intended to contribute towards its flexural strength, its position within the slab is not important except that it should be adequately protected from corrosion. Since cracks starting with higher tensile stress at the top surface are more critical when they tend to open, the general preference is for the placing of reinforcement about 50 mm below the surface. Reinforcement is often continued across dummy groove joints to serve the same purpose as tie bars, but at all full depth joints it is kept at least 50mm away from the face of the joint or edge.

The reinforcement should have 60mm cover from the surface except for slab less than 150mm thick where 50mm cover should be provided.

8.2.4 Joints and Joint Spacing

The expansion, contraction and construction joints should be as per details shown in **Figure 8-5** and these joints may be with or without dowel bars. The recommended spacing of joints and requirements of dowel bars in Rigid pavement for Highways are shown in **Table 8.2, Table 8.3 & Table 8.4.**

a) Expansion joint spacing (For 25mm wide Expansion joints)

TABLE 8.2

Period of Construction	Degree of foundation roughness	Maximum Expansion joint spacing (m) Slab Thickness (mm)			
		100	150	200	250
Winter (October-March)	Smooth	40	40	40	60
	Rough	100	120	120	120
Summer (April-September)	Smooth	50	60	60	100
	Rough	100	120	120	120

b) Construction/Contracting Joint Spacing

TABLE 8.3

Slab Thickness (mm)	Maximum Contraction Joint Spacing (m)
Unreinforced Slab	
150	4.5
150	4.5
200	4.5
Reinforced Slabs	
100	7.5
150	13.0
200	14.0

c) Dimensions of dowel bars for Expansion & Contraction joints

TABLE 8.4

Slab Thickness (mm)	Expansion joints		Contraction joints	
	Diameter (mm)	Length (mm)	Diameter (mm)	Length (mm)
150 - 180	20	500	12	400
190 - 230	25	500	20	500

Note-1, The spacing of dowel bars should be 200mm for 20mm dia bar & 300mm for 25mm dia bar.

Note-2, Dowel bars are not recommended for slabs thinner than 150mm.

Typical tie bar details for use at central longitudinal joint in double-lane rigid pavements with a lane width of 3.50m are given in **Table 8.5**.

TABLE 8.5

Details of Tie bars for central longitudinal joint in two-lane rigid highway pavement.

Slab thickness mm	Tiebar details			
	Diameter (mm)	Maximum spacing (mm)	Minimum Length (mm)	
			Plain bar	Deformed bars
150	8	380	400	300
	10	600	450	350
200	10	450	450	350
	12	640	550	400

Example :

Concrete Slab with Reinforcement

1. Design life of the Pavement = 20 years.
2. Cumulative Equivalent Standard Axles = 2 million
3. Normal Subgrade - CBR 8%

The subgrade & concrete thickness will be as follows :

(a) Thickness of Sub base - 80 mm (**Table 8.1**)
(b) Concrete slab Thickness - 180mm (both reinforced & unreinforced) : **Fig. 8-1**

From **Fig. 8-3** minimum reinforcements will be 2.61 Kg/m^2 or 275mm^2/m width of slab for ESA of 2 million and from **Fig. 8-4** and the maximum spacing of joints may be selected as 16.5 meter. The reinforcement of 10mm dia bar at an interval of 300 mm shall be placed longitudinally 60mm below the top surface of the slab. The transverse reinforcement may be 10mm dia bar 450mm interval just as a binder to the main reinforcement. **Figure 8-6** and **8-7** show typical cross-sections of slab with reinforcements and without reinforcements respectively.

According to the formula, the reinforcement may be calculated as follows :

$$A = \frac{LFW}{2S}$$

A (Longitudinal) = $\frac{13 \times 1.5 \times 360}{2 \times 0.5 \times 2800}$ = 2.51 cm^2; use 10mm dia @ 270mm c/c

A (Transverse) = $\frac{3.6 \times 1.5 \times 360}{2. \times 0.5 \times 2800}$ = 0.69 cm^2 ; use 10mm dia @ 300mm c/c

(c) Expansion Joint : This may be selected from **Table 8.2**. For thickness of slab 150mm & 200mm, the Expansion Joint will be 120 meter.

(d) Construction & Contraction Joint : The spacing of construction or contraction joint will be 4.5m from Table 8-3 for unreinforced slab & 13 meter for reinforced slab.

(e) The dowel bars in concrete slab : From **Table 8.4** for construction joint the length of dowel bar shall be 400mm & diameter of 12mm.

For Expansion joint the length of dowel bar shall be 500 mm & dia will be 20 mm.

Unreinforced Slab - 180mm thickness

Contraction/Construction Joint - Type 1 & 2 of **Fig. 8-5.**

Expansion joint - **Fig. 8-5.**

For dowel bars if provided, then it will be as shown in **Fig. 8-5.**

Reinforced Slab - 180 mm thickness .

The joints will be as shown in **Fig. 8-5.**

The reinforcement in Longitidinal/Transverse direction shall be as follows :

(1) As per Road Note 29 :

(2) As per formula stated in this example :
The traffic data for the design life of Pavement is shown here for single & double lane Road.

APPENDICES

* Appendix - A : THE CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES FOR HIGHWAYS CONSTRUCTION PURPOSES

* Appendix - B : MATERIALS FOR AGGREGATE AND SOIL-AGGREGATE SUB-BASE, BASE AND SURFACES COURSES

* Appendix - C : ROAD NOTE 31 (FOURTH EDITION) : A GUIDE TO THE STRUCTURAL DESIGN OF BITUMEN-SURFACED ROADS IN TROPICAL AND SUB-TROPICAL COUNTRIES

* Appendix - D : DRAINAGE AND ROAD SHOULDERS

* Appendix - E : UNBOUND PAVEMENT MATERIALS

* Appendix - F : STRUCTURE CATALOGUE

* Appendix - G : EXAMPLE ON TRAFFIC DATA COLLECTION, TRAFFIC ANALYSIS AND PAVEMENT DESIGN

* Appendix - H : COMPOSITION OF THE TECHNICAL WORKING GROUP

Road Note 31 (Fourth Edition)

A GUIDE TO THE STRUCTURAL DESIGN OF BITUMEN-SURFACED ROADS IN TROPICAL AND SUB-TROPICAL COUNTRIES

1.0 INTRODUCTION

1.1 General

This Road Note gives recommendations for the structural design of bituminous surfaced roads in tropical and sub-tropical climates. It is aimed at highway engineers responsible for the design and construction of new road pavements and is appropriate for roads which are required to carry up to 30 million cumulative equivalent standard axles in one direction. The design of strengthening overlays is not covered nor is the design of earth, gravel or concrete roads. Although this Note is appropriate for the structural design of flexible roads in urban areas, some of the special requirements of urban roads, such as the consideration of kerbing, sub-soil drainage, skid resistance etc. are not covered.

For the structural design of more heavily trafficked roads, the recommendations of this Note may be supplemented by those given in the guides for the design of bituminous pavements in the United Kingdom (Powell et al (1984)) but these are likely to require some form of calibration or adaptation to take account of the conditions encountered in the tropics.

1.2 Road Deterioration

The purpose of structural design is to limit the stresses induced in the subgrade by traffic to a safe level at which subgrade deformation is insignificant whilst at the same time ensuring that the road pavement layers themselves do not deteriorate to any serious extent within a specified period of time.

By the nature of the materials used for construction, it is impossible to design a road pavement which does not deteriorate in some way with time and traffic, hence the aim of structural design is to limit the level of pavement distress, measured primarily in terms of riding quality, rut depth and cracking, to predetermined values. Generally these values are set so that a suitable remedial treatment at the end of the design period is a strengthening overlay of some kind but this is not necessarily so and roads can, in principle, be designed to reach a terminal condition at which major rehabilitation or even complete reconstruction is necessary. However, assessing appropriate remedial treatments for roads which have deteriorated beyond a certain level is a difficult task. In most design methods it is assumed that adequate routine and periodic maintenance is carried out during the design period of the road and that at the end of the design period a relatively low level of deterioration has occurred.

Acceptable levels of surface condition have usually been based on the expectations of road users. These expectations have been found to depend upon the class of road and the volume of traffic such that the higher the geometric standard, and therefore the higher the vehicle speeds, the lower the level of pavement distress which is acceptable. In defining these levels, economic considerations were not considered because there was insufficient knowledge of the cost trade-offs for an economic analysis to be carried out with sufficient accuracy.

1.3 Economic Considerations

In recent years a number of important empirical studies have shown how the costs of operating vehicles depend on the surface condition of the road. The studies have also improved our knowledge of how the deterioration of roads depends on the nature of the traffic, the properties of the road-making materials, the environment, and the maintenance strategy adopted (Parsley and Robinson (1982). Paterson (1987), Chesher and Harrison (1987), Watanatada et al (1987)). In some circumstances it is now possible to design a road in such a way that provided maintenance and strengthening can be carried out at the proper time, the total cost of the transport facility i.e. the sum of construction costs, maintenance costs and road user costs, can be minimised. These techniques are expected to become more widespread in the future. Also with the introduction in many countries of pavement management systems in which road condition is monitored on a regular basis, additional information will be collected to allow road performance models to be refined. Pavement structural design could then become an integral part of the management system in which design could be modified according to the expected maintenance inputs in such a way that the economic strategies could be adopted. Whilst these refinements lie in the future, the research has provided important guidance on structural designs suitable for tropical and sub-tropical environments and has been used, in part in preparing this edition of Road Note 31.

For the structures recommended in this Note, 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 has been assumed that routine and periodic maintenance activities are carried out to a reasonable, through not excessive, level. In particular, it has been assumed that periodic maintenance is done whenever the area of road surface experiencing defects i.e. cracking, ravelling etc., exceeds 15 per cent. For example, for a 10 year design period, one surface maintenance treatment is likely to be required for the higher traffic levels whereas for a 15 years design period, one treatment is likely to be required for the lower traffic levels and two for the higher. These are broad guidelines only and the exact requirements will depend on local conditions.

1.4 Effects of Climate

Research has shown how different types of road deteriorate and has demonstrated that some of the most common modes of failure in the tropics are often different from those encountered in temperate regions. In particular, climate related deterioration sometimes dominates performance and the research has emphasised the overriding importance of the design of bituminous surfacing materials to minimise this type of deterioration (Paterson (1987). Smith et al (1990). Strauss et al (1984)). This topic is dealt with in Chapter 8.

Climate also affects the nature of the soils and rocks encountered in the tropics. Soil-forming processes are still very active and the surface rocks are often deeply weathered. The soils themselves often display extreme or unusual properties which can pose considerable problems for road designers. The recent publication 'Road building in the tropics: materials and methods' provides an introduction to these topics (Millard (1993)).

1.5 Variability in Material Properties and Road

Variability in material properties and construction control is generally much greater than desired by the design engineer and must be taken into account explicitly in the design process. Only a very small percentage of the area of the surface of a road needs to show distress for the road to be considered unacceptable by road users. It is therefore the weakest parts of the road or the extreme tail of the statistical distribution of 'strength' which is important in design. In well controlled full-scale experiments this variability is such that the ten per cent of the road which performs best will carry about six times more traffic before reaching a defined terminal condition than the ten per cent which performs least well. Under normal construction conditions this spread of performance becomes even greater. Some of this variability can be explained through the measured variability of those factors known to affect performance. Therefore, if the likely variability is known beforehand, it is possible, in principle, for it to be taken into account in design. It is false economy to minimise the extent of preliminary investigations to determine this variability.

In practice it is usually only the variability of subgrade strength that is considered and all other factors are controlled by means of specifications i.e. by setting minimum acceptable values for the key properties. But specifications need to be based on easily measurable attributes of the materials and these may not correlate well with the fundamental mechanical properties on which behaviour depends. As a result, even when the variability of subgrade strength and pavement material properties are taken into account, there often remains a considerable variation in performance between nominally identical pavements which cannot be fully explained. Optimum design therefore remains partly dependent on knowledge of the performance of in-service roads and quantification of the variability of the observed performance itself. Thus there is always likely to be scope for improving designs based on local experience.

Nevertheless, it is the task of the designer to estimate likely variations in layer thicknesses and material strengths so that realistic target values and tolerances can be set in the specifications to ensure that satisfactory road performance can be guaranteed as far as is possible.

The thickness and strength values described in this Road Note are essentially minimum values but practical considerations require that they are interpreted as lower ten percentile values with 90 per cent of all test results exceeding the values quoted. The random nature of variations in thickness and strength which occur when each layer is constructed should ensure that minor deficiencies in thickness or strength do not occur one on top of the other, or very rarely so. The importance of good practice in quarrying. material handling and stockpiling to ensure this randomness and also in minimise variations themselves cannot be over emphasised.

1.6 Uncertainty in Traffic Forecasts

Pavement design also depends on the expected level of traffic. Axle load studies and traffic counts are essential prerequisites for successful design but traffic forecasting remains a difficult task and therefore sensitivity and risk analysis and recommended. This topic is discussed in Chapter 2.

1.7 Basis for the Design Catalogue

The pavement designs incorporated into the fourth edition of Road Note 31 are based primarily on :

(a) The results of full-scale experiments where all factors affecting performance have been accurately measured and their variability quantified.

(b) Studies of the performance of as-built existing road networks.

Where direct empirical evidence is lacking, designs have been interpolated or extrapolated from empirical studies using road performance models (Parsley and Robinson (1982), Paterson (1987), Rolt et al (1987)) and standard analytical, mechanistic methods e.g. Gerritsen and Koole (1987), Powell et al (1984), Brunton et al (1987).

In view of the statistical nature of pavement design caused by the large uncertainties in traffic forecasting and the variability in material properties, climate and road behaviour, the design charts have been presented as a catalogue of structures, each structure being applicable over a small range of traffic and subgrade strength. Such a procedure makes the charts extremely easy to use but it is important that the reader is thoroughly conversant with the notes applicable to each chart.

Throughout the text the component layers of a flexible pavement are referred to in the following terms (see **Figure 1**).

Surfacing : This is the uppermost layer of the pavement and will normally consist of a bituminous surface dressing or a layer of premixed bituminous material. Where premixed materials are laid in two layers, these are known as the wearing course and the basecourse (or binder course) as shown in **Figure 1**.

Roadbase : This is the main load-spreading layer of the pavement. It will normally consist of crushed stone or gravel, or of gravelly soils, decomposed rock, sands and sand-clays stabilised with cement, lime or bitumen.

Sub-base : This is the secondary load-spreading layer underlying the roadbase. It will normally consist of a material of lower quality than that used in the roadbase such as unprocessed natural gravel, gravel-sand, or gravel-sand-clay. This layer also serves as a separating layer preventing contamination of the roadbase by the subgrade material and under wet conditions, it has an important role to play in protecting the subgrade from damage by construction traffic.

Capping layer (selected or improved subgrade): Where very weak soils are encountered, a capping layer is sometimes necessary. This may consist of better quality subgrade material imported from elsewhere or existing subgrade material improved by mechanical or chemical stabilisation.

Subgrade : This is the upper layer of the natural soil which may be undisturbed local material or may be soil excavated elsewhere and placed as fill. In either case it is compacted during construction to give added strength.

1.8 The Design Process

There are three main steps to be followed in designing a new road pavement. These are :

(i) estimating the amount of traffic and the cumulative number of equivalent standard axles that will use the road over the selected design life;

(ii) assessing the strength of the subgrade soil over which the road is to be built;

(iii) selecting the most economical combination of pavement materials and layer thicknesses that will provide satisfactory service over the design life of the pavement. (It is usually necessary of assume that an appropriate level of maintenance is also carried out).

This Note considers each of these steps in turn and puts special emphasis on five aspects of design that are of major significance in designing roads in most tropical countries :

- The influence of tropical climates on moisture conditions in road subgrades.
- The severe conditions imposed on exposed bituminous surfacing materials by tropical climates and the implications of this for the design of such surfacings.
- The interrelationship between design and maintenance. If an appropriate level of maintenance cannon be assumed, it is not possible to produce designs that will carry the anticipated traffic loading without high costs to vehicle operators through increased road deterioration.
- The high axle loads and tyre pressures which are common in most countries.
- The influence of tropical climates on the nature of the soils and rocks used in road building.

The overall process of designing a road is illustrated in Figure 2. Some of the information necessary to carry out the tasks may be available from elsewhere e.g. a feasibility study or Ministry records, but all existing data will need to be checked carefully to ensure that it is both up-to-date and accurate. Likely problem areas are highlighted in the relevant chapters of this Note.

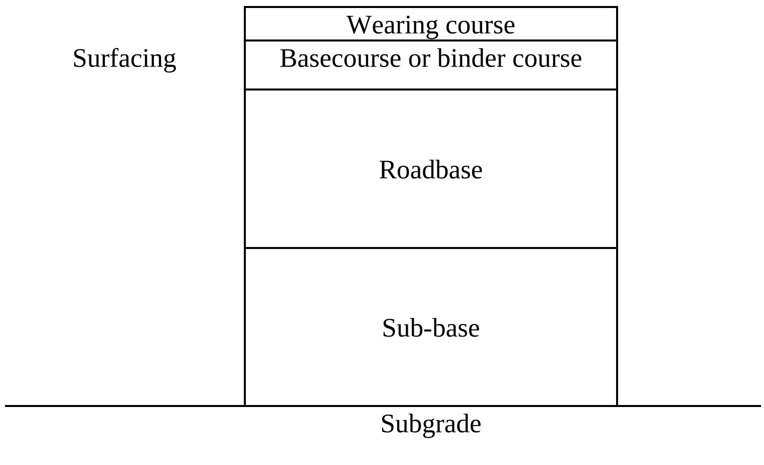

Fig. 1 Definition of Pavement Layers

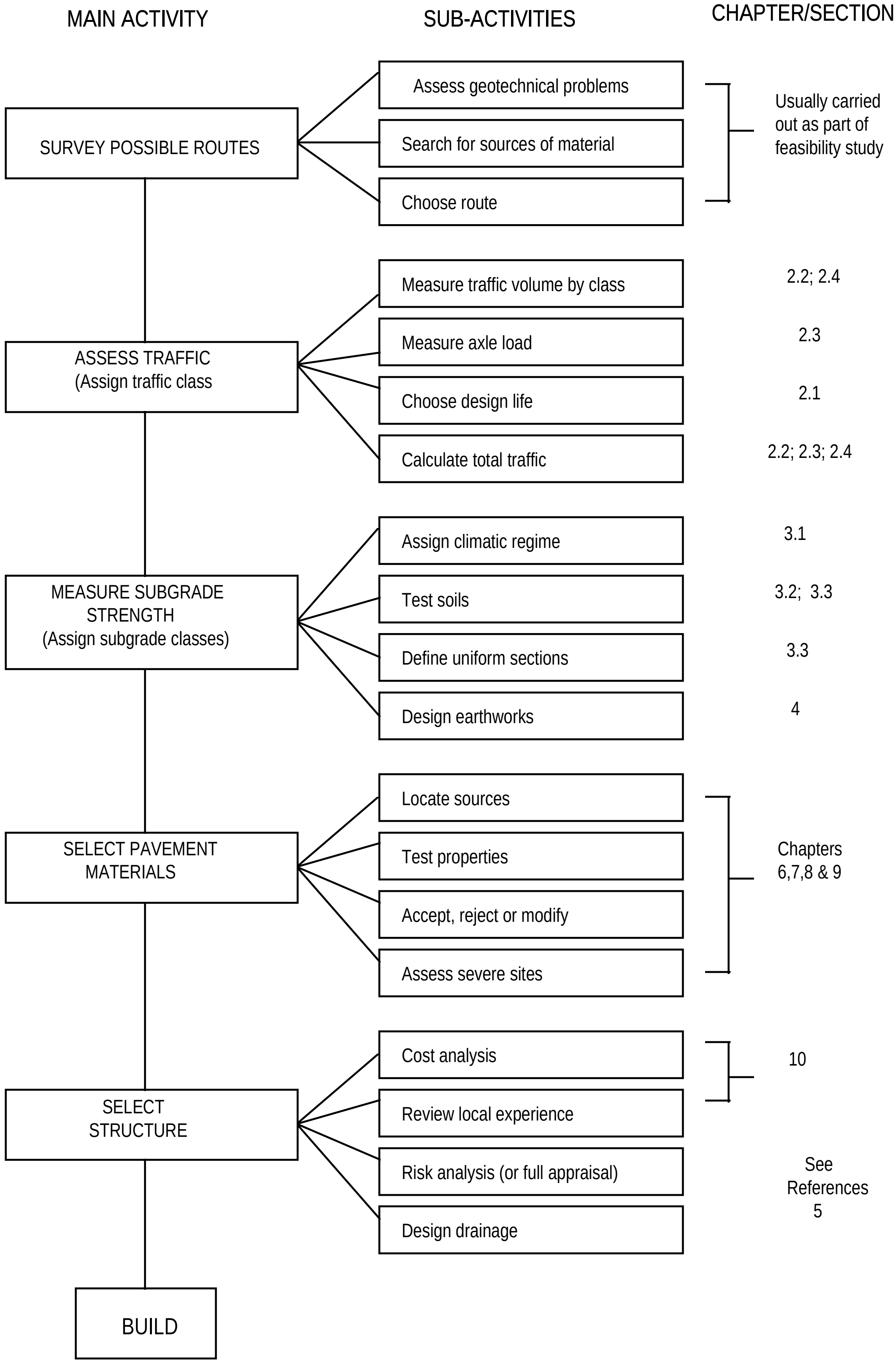

Fig.2 The pavement design process

2. TRAFFIC

The deterioration of paved roads caused by traffic results from both the magnitude of the individual wheel loads and the number of times these loads are applied. For pavement design purposes it is necessary to consider not only the total number of vehicles that will use the road but also the wheel loads (or, for convenience, the axle loads) of these vehicles. The loads imposed by private cars do not contribute significantly to the structural damage. For the purposes of structural design, cars and similar sized vehicles can be ignored and only the total number and the axle loading of the heavy vehicles that will use the road during its design life need to be considered. In this context, heavy vehicles are defined as those having an unladen weight of 3000 kg or more. In some circumstances, particularly for lightly trafficked roads, construction traffic can be a significant component of overall traffic loading and the designs should take this into account.

2.1 Design Life

For most road projects an economic analysis period of between 10 and 20 years from the date of opening is appropriate, but for major projects this period should be tested as part of the appraisal process (Overseas Road Note 5, Transport and Road Research Laboratory (1988)). Whatever time period is chosen for the appraisal of a project, the road will always have some residual value at the end of this period. Choosing a pavement design life that is the same as the analysis period simplifies the economic appraisal by minimising the residual value, which is normally difficult to estimate accurately. A pavement design life of 15 years also reduces the problem of forecasting uncertain traffic trends for long periods into the future.

In this context, 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. Condition surveys of bituminous pavements should be carried out about once a year as part of the inspection procedures for maintenance. These are used to determine not only the maintenance requirements but also the nature and rate to change of condition to help to identify if and when the pavement is likely to need strengthening.

Stage construction consists of planned improvements to the pavement structure at fixed times through the project life. From a purely economic point of view, stage construction policies have much to commend them. However, experience has shown that budget constraints have often prevented the planned upgrading phases of stage construction projects from taking place, with the result that much of the benefit from such projects has been lost. In general, stage construction policies are not recommended if there is any risk that maintenance and upgrading will not be carried out correctly or at the appropriate time.

2.2 ESTIMATING TRAFFIC FLOWS

2.2.1 Baseline traffic flows

In order to determine the total traffic over the design life of the road, the first step is to estimate baseline traffic flows. The estimate should be the (Annual) Average Daily Traffic (ADT) currently using the route, classified into the vehicle categories of cars, light goods vehicles, trucks (heave goods vehicles) and buses. The ADT is defined as the total annual traffic summed for ***both*** directions and divided by 365. It is usually obtained by recording actual traffic flows over a shorter period from which the ADT is then estimated. For long projects, large differences in traffic along the road may make it necessary to estimate the flow at several locations. It should be noted that for structural design purposes the traffic loading in ***one*** direction is required and for this reason care is always required when interpreting ADT figures.

Traffic counts carried out over a short period as a basis for estimating the traffic flow can produce estimates which are subject to large errors because traffic flows can have large daily, weekly, monthly and seasonal variations (Howe (1972)). The daily variability in traffic flow depends on the volume of traffic. It increases as traffic levels fall, with high variability on roads carrying less than 1000 vehicles per day. Traffic flows vary more from day-to-day than from week-to-week over the year. Thus there are large errors associated with estimating average daily traffic flows (and subsequently annual traffic flows) from traffic counts of only a few days duration, or excluding the weekend. For the same reason there is a rapid decrease in the likely error as the duration of the counting period increases up to one week. For counts of longer duration, improvements in accuracy are less pronounced. Traffic flows also vary from month-to-month so that a weekly traffic count repeated at intervals during the year provides a better base for estimating the annual volume of traffic than a continuous traffic count of the same duration. Traffic also varies considerably through a 24-hour period and this needs to be taken into account explicitly as outlined below :

In order to reduce error, it is recommended that traffic counts to establish ADT at a specific conform to the following practice :

(i) The counts are for seven consecutive days .

(ii) The counts on some of the days are for a full 24 hours, with preferably at least one 24-hour count on a weekday and one during a weekend. On the other days 16-hour counts should be sufficient. These should be grossed up to 24-hour values in the same proportion as the 16-hour/24-hour split on those days when full 24-hour counts have been undertaken.

(iii) Counts are avoided at times when travel activity is abnormal for short periods due to the payment of wages and salaries, public holidays, etc. If abnormal traffic flows persist for extended periods, for example during harvest times, additional counts need to be made to ensure this traffic is properly included.

(iv) If possible, the seven-day counts should be repeated several times throughout the year.

Country-wide traffic data should be collected on a systematic basis to enable seasonal trends in traffic flows to be quantified. Unfortunately, many of the counts that are available are unreliable, especially it then have been carried out manually. Therefore, where seasonal adjustment factors are applied to traffic survey data in order to improve the accuracy of baseline traffic figures, the quality of the statistics on which they are based should be checked in the field.

Classified traffic counts are normally obtained by counting manually. These counts can be supplemented by automatic counters which use either a pneumatic tube laid across the surface of the carriageway or a wire loop fixed to the carriageway surface or, preferably, buried just beneath it. Pneumatic tubes require regular maintenance and can be subject to vandalism. Buried loops are preferred, but installing a loop beneath the road surface is more difficult and more expensive than using a pneumatic tube.

In their basic form, automatic counters do not distinguish between difficult types of vehicle and so cannot provide a classified count. Modern detector systems are now available which can perform classified vehicle counting, but such systems are expensive and not yet considered to be sufficiently robust for most developing country applications. An exception is the counter system developed specifically for developing countries by the Transport Research Laboratory.

2.2.2 Traffic Forecasting

Even with a developed economy and stable economic conditions, traffic forecasting is an uncertain process. In a developing economy the problem becomes more difficult because such economies are often sensitive to the world prices of just one or two commodities.

In order to forecast traffic growth it is necessary to separate traffic into the following three categories :

(a) ***Normal Traffic :*** Traffic which would pass along the existing road or track even if no new pavement were provided.

(b) ***Diverted Traffic :*** Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.

(c) ***Generated Traffic :*** Additional traffic which occurs in response to the provision or improvement of the road.

Normal Traffic : The commonest method of forecasting normal traffic is to extrapolate time series data on traffic levels and assume that growth will either remain constant in absolute terms i.e. a fixed number of vehicles per year (a linear extrapolation), or constant in relative terms i.e. a fixed percentage increase. Data in fuel sales can often be used as a guide to country-wide growth in traffic levels, although improvements in fuel economy over time should be taken into account. As a general rule it is only safe to extrapolate forward for as many years as the same general economic conditions are expected to continue.

As an alternative to time, growth can be related linearly to anticipated Gross Domestic Product (GDP). This is normally preferable since it explicitly takes into account changes in overall economic activity, but it has the disadvantage that a forecast of GDP's is needed. The use of additional variables, such as population or fuel price, brings with it the same problem. If GDP forecasts are not available, then future traffic growth should be based on time series data.

If it is thought that a particular component of the traffic will grow at a different rate to the rest, it should be specifically identified and dealt with separately. For example, there may be a plan to expand a local town or open a local factory during the design life of the road, either of which could lead to different growth rates for different types of vehicle, or there may be a plan to allow larger freight vehicles on the road, in which case the growth rate for trucks may be relatively low because each truck is heavier.

Whatever the forecasting procedure used, it is essential to consider the realism of forecast future levels. Few developing countries are likely to sustain the high rates of growth experienced in the past, even in the short term, and factors such as higher fuel costs and vehicle import restrictions could tend to depress future growth rates.

Diverted Traffic : Where parallel routes exist, traffic will usually travel on the quickest or cheapest route although this may not necessarily be the shortest. Thus, surfacing an existing road may divert traffic from a parallel and shorter route because higher speeds are possible on the surfaced road. Origin and destination surveys should be carried out to provide data on the traffic diversions likely to arise. Assignment of diverted traffic is normally done by an all-or-nothing method in which it is assumed that all vehicles that would save time or money by diverting would do so, and that vehicles that would lose time or increase costs would not transfer. With such a method it is important that all perceived costs are included. In some of the more developed countries there may be scope for modelling different scenarios using standard assignment computer programs.

Diversion from other transport modes, such as rail or water, is not easy to forecast. Transport of bulk commodities will normally be by the cheapest mode, though this may not be the quickest. However, quality of service, speed and convenience are valued by intending consignors and, for general goods, diversion from other modes should not be estimated solely on the basis of door-to-door transport charges. Similarly, the choice of mode for passenger transport should not be judged purely on the basis of travel charges. The importance attached to quality of service by users has been a major contributory factor to the worldwide decline in rail transport over recent years.

Diverted traffic is normally forecast to grow at the same rate as traffic on the road from which it diverted.

Generated Traffic : Generated traffic arises either because a journey becomes more attractive by virtue of a cost or time reduction or because of the ***increased*** development that is brought about by the road investment. Generated traffic is difficult to forecast accurately and can be easily overestimated. It is only likely to be significant in those cases where the road investment brings about large reductions in transport costs. For example, in the case of a small improvement within an already developed highway system, generated traffic will be small and can normally be ignored. 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, for example, animal-based transport to motor vehicle transport. In such a case, generated traffic could be the main component of future traffic flow.

The recommended approach to forecasting generated traffic is to use demand relationships. The price elasticity of demand for transport is the responsiveness of traffic to a change in transport costs following a road investment. On inter-urban roads a distinction is normally drawn between passenger and freight traffic. On roads providing access to rural areas, a further distinction is usually made between agricultural and non-agricultural freight traffic.

Evidence from several evaluation studies carried out in developing countries gives 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 one per cent decrease in transport costs leads to a one per cent increase in traffic. Calculations should be based on door-to-door travel costs estimated as a result of origin and destination surveys and not just on that part of the trip incurred on the road under study.

The available evidence suggests that the elasticity of demand for passenger travel is usually slightly greater than unity. In general, the elasticity of demand for goods is much lower and depends on the proportion of transport costs in the commodity price.

2.3 AXLE LOADING

2.3.1 Axle Equivalency

The damage that vehicles do to a road depends very strongly on the axle loads of the vehicles. For pavement design purposes the damaging power of axles is related to a 'standard' axle of 8.16 tonnes using equivalence factors which have been derived from empirical studies (Highway Research Board (1962), Paterson (1987). In order to determine the cumulative axle load damage that a pavement will sustain during its design life, it is necessary to express the total number of heavy vehicles that will use the road over this period in terms of the cumulative number of equivalent standard axles (esa).

Axle load surveys must be carried out to determine the axle load distribution of a sample of the heavy vehicles using the road. Data collected from these surveys are used to calculate the mean number of equivalent standard axles for a typical vehicle in each class. These values are then used in conjunction with traffic forecasts to determine the predicted cumulative equivalent standard axles that the road will carry over its design life.

2.3.2 Axle Load Surveys

If no recent axle load data are available it is recommended that axle load surveys of heavy vehicles are undertaken whenever a major road project is being designed. Ideally, several surveys at periods which will reflect seasonal changes in the magnitude of axle loads are recommended. Portable vehicle-wheel weighing devices are available which enable a small team to weigh up to 90 vehicles per hour. Detailed guidance on carrying out axle load surveys and analysing the results is given in TRRL. Road Note 40 (Transport and Road Research Laboratory (1978)).

It is recommended that axle load surveys are carried out by weighing a sample of vehicles at the roadside. The sample should be chosen such that a maximum of about 60 vehicles per hour are weighed. The weighing site should be level and, if possible, constructed in such a way that vehicles are pulled clear of the road when being weighed. The portable weighbridge should be mounted in a small pit with its surface ***level*** with the surrounding area. This ensures that all of the wheels of the vehicle being weighed are level and eliminates the errors which can be introduced by even a small twist or tilt of the vehicle. More importantly, it also eliminates the large errors that can occur if all the wheels on one side of multiple axle groups are not kept in the same horizontal plane. The load distribution between axles in multiple axle groups is often uneven and therefore each axle must be weighed separately. The duration of the survey should be based on the same considerations as for traffic counting outlined in Section 2.2.1.

On certain roads it may be necessary to consider whether the axle load distribution of the traffic travelling in one direction is the same as that of the traffic travelling in the opposite direction. Significant differences between the two streams can occur or roads serving docks, quarries, cement works, etc., where the vehicles travelling one way are heavily loaded but are empty on the return journey. In such cases the result from the more heavily trafficked lane should be used when converting commercial vehicle flows to the equivalent number of standard axles for pavement design. Similarly, special allowance must be made for unusual axle loads on roads which mainly serve one specific economic activity, since this can result in a particular vehicle type being predominant in the traffic spectrum. This is often the case, for example, in timber extraction areas, mining areas and oil fields.

2.3.3 Determination of cumulative equivalent standard axles

Computer programs have been written to assist with the analysis of the results from axle load surveys. These programs provide a detailed tabulation of the survey results and determine the mean equivalence factors for each vehicle type if required. If such a program is not available, standard spreadsheet programs can be used.

If there are no computer facilities available the following method of analysis is recommended. The equivalence factors for each of the wheel loads measured during the axle load survey are determined using Table 2.1 or the accompanying equation to obtain the equivalence factors for vehicle axles. The factors for the axles are totalled to give the equivalence factor for each of the vehicles. For vehicles with multiple axles i.e. tandems, triples etc., each axle in the multiple group is considered separately.

The mean equivalence factor for each type or class of vehicle travelling in each direction must then be determined. Vehicle classes are usually defined by the number and type of axles. Note that this method of determining the mean equivalence factors must always be used; calculating the equivalence factor for the average axle load is incorrect and leads to large errors.

In order to determine the cumulative equivalent standard axles over the design life of the road, the following procedure should be followed.

(i) Determine the daily traffic flow for each class of vehicle weighed using the results of the traffic survey and any other recent traffic count information that is available.

(ii) Determine the average daily one-directional traffic flow for each class of vehicle.

(iii) Make a forecast of the one-directional traffic flow for each class of vehicle to determine the total traffic in each class that will travel over each lane during the design life (see Section 2.2.2.).

(iv) Determine the mean equivalence factor for each class of vehicle and for each direction from the results of this axle load survey and any other surveys that have recently been carried out.

(v) The products of the cumulative one-directional traffic flows for each class of vehicle over the design life of the road and the mean equivalence factor for that class should then be calculated and added together to give the cumulative equivalent standard axle loading for each direction. The higher of the two directional values should be used for design.

TABLE 2.1

Equivalence factors for different axle loads

Wheel load (single & dual) (10^3 kg)	Axle load (10^3 kg)	Equivalence factor
1.5	3.0	0.01
2.0	4.0	0.04
2.5	5.0	0.11
3.0	6.0	0.25
3.5	7.0	0.50
4.0	8.0	0.91
4.5	9.0	1.55
5.0	10.0	2.50
5.5	11.0	3.83
6.0	12.0	5.67
6.5	13.0	8.13
7.0	14.0	11.3
7.5	15.0	15.5
8.0	16.0	20.7
8.5	17.0	27.2
9.0	18.0	35.2
9.5	19.0	44.9
10.0	20.0	56.5

$$\textbf{Equivalence Factor} = \left(\frac{\textbf{Axleload(kg)}}{8160}\right)^{4.5}$$

In most countries the axle load distribution of the total population of heavy vehicles using the road system remains roughly constant from year to year although there may be long-term trends resulting from the introduction of new types of vehicles or changes in vehicle regulations and their enforcement. It is therefore customary to assume that the axle load distribution of the heavy vehicles will remain unchanged for the design life of the pavement and that it can be determined by undertaking surveys of vehicle axle loads on existing roads of the same type and which serve the same function. In most developing countries the portable errors in these assumptions for a design life of 15 years are unlikely to result in a significant error in design.

On dual carriageway roads and on single carriageway roads with more than two lanes, it should be assumed that the slow traffic lanes will carry all the heavy vehicles unless local experience indicates otherwise or the traffic flow exceeds about 2000 heavy vehicles per day in each direction. In the latter case, a proportion of heavy vehicles should be assigned to the slow lave according to the principles outlines in Overseas Road Note No. 6 (Transport and Road Research Laboratory (1988)). The design thickness required for the slow lane is usually applied to the whole carriageway width but there may be situations where a tapered roadbase or sub-base is appropriate.

In some countries, single-lane bituminous roads are built to economise on construction costs. On such roads the traffic tends to be more channelised than on two-lane roads. The effective traffic loading in the wheelpath in one direction has been shown to be ***twice*** that for a wider road. Therefore, taking into account the traffic in both directions, the pavement thickness for these roads should be based on ***four*** times the total number of heavy vehicles that travel in one direction.

2.4 Accuracy

All survey data are subject to errors. Traffic data, in particular, can be very inaccurate and predictions about traffic growth are also prone to large errors. Accurate calculations of cumulative traffic are therefore very difficult to make. To minimise these errors there is no substitute for carrying out specific traffic surveys for each project for the durations suggested in Section 2.2.1. Additional errors are introduced in the calculation of cumulative standard axles because any small errors in measuring axle loads are amplified by the fourth power law relationship between the two.

Fortunately, pavement thickness design is relatively insensitive to cumulative axle load and the method recommended in this Note provides fixed structures for ranges of traffic as shown in **Table 2.2**. As long as the estimate of cumulative equivalent standard axles is close to the centre of one of the ranges, any errors are unlikely to affect the choice of pavement design. However, if estimates of cumulative traffic are close to the boundaries of the traffic ranges then the basic traffic data and forecasts should be re-evaluated and sensitivity analyses carried out to ensure that the choice of traffic class is appropriate. Formal risk analysis can also be used to evaluate the design choices as described briefly and referenced in Overseas Road Note
5 (Transport and Road Research Laboratory (1988)).

TABLE 2.2

Traffic Classes

Traffic classes	Range (10^6 esa)
T1	< 0.3
T2	0.3-0.7
T3	0.7-1.5
T4	1.5-3.0
T5	3.0-6.0
T6	6.0-10
T7	10-17
T8	17-30

3. THE SUBGRADE

The type of subgrade soil is largely determined by the location of the road, but where the soils within the possible corridor for the road vary significantly in strength from place to place, it is clearly desirable to locate the pavement on the stronger soils if this does not conflict with other constraints.

The strength of road subgrades is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content.

For designing the thickness of a road pavement, the strength of the subgrade should be taken as that of the soil at a moisture content equal to the wettest moisture condition likely to occur in the subgrade after the road is opened to traffic. In the tropics, subgrade moisture conditions under ***impermeable*** road pavements can be classified into three main categories:

Category (1): Subgrades where 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 becomes the dominant influence on the subgrade moisture content. For example, in non-plastic soils the water table will dominate the subgrade moisture content when is rises to within 1m of the road surface, in sandy clays (PI<20 per cent) the water table will dominate when it rises to within 3m of the road surface, and in heavy clays (PI>40 per cent) the water table will dominate when if rises to within 7m of the road surface.

In addition to areas where the water table is maintained by rainfall, this category includes coastal strips and flood plains where the water table is maintained by the sea, by a lake or by a river.

Category (2): Subgrades with deep water tables and where rainfall is sufficient to produce significant changes in moisture conditions under the road.

These conditions occur when rainfall exceeds evapotranspiration for at least two months of the year. The rainfall in such areas is usually greater than 250mm per year and is often seasonal.

Category (3): Subgrades in areas with no permanent water table near the ground surface and where the climate is dry throughout most of the year with an annual rainfall of 250 mm or less.

Direct assessment of the likely strength or CBR of the subgrade soil is often difficult to make but its value can be inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density and moisture content for the soil in question. The relationship must be determined in the laboratory. The density of the subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction. The moisture content of the subgrade soil is governed by the local climate and the depth of the water table below the road surface. In most circumstances, the first task is therefore to estimate the equilibrium moisture content as outlined in Section 3.1 below. A method of direct assessment of the subgrade strength, where this is possible, is discussed in Section 3.2 together with less precise methods of estimation which can be used if facilities for carrying out the full procedure are not available.

3.1 Estimating the Subgrade Moisture Content

Category (1): The easiest method of estimating the design subgrade moisture content is to measure the moisture content in subgrades below existing pavements in similar situations at the time of the year when the water table is at its highest level. These pavements should be greater than 3m wide and more than two years old and samples should preferably be taken from under the carriageway about 0.5m from the edge. Allowance can be made for different soil types by virtue of the fact that the ratio of subgrade moisture content to plastic limit is the same for different subgrade soils when the water table and climatic conditions are similar. If there is no suitable road in the vicinity, the moisture content in the subgrade under an impermeable pavement can be estimated from a knowledge of the depth of the water table and the relationship between suction and moisture content for the subgrade soil (Russam and Croney (1960)). The test apparatus required for determining this relationship is straightforward and the method is described in Appendix B.

Category (2): When the water table is not near the ground surface, the subgrade moisture condition under an impermeable pavement will depend on the balance between the water entering the subgrade through the shoulders and at the edges of the pavement during wet weather and the moisture leaving the ground by evapotranspiration during dry periods. Where the average annual rainfall is greater than 250mm a year, the moisture condition for design purposes can be taken as the optimum moisture content given by the British Standard (Light) Compaction Test, 2.5 kg rammer method.

When deciding on the depth of the water table in Category (1) or Category (2) subgrades, the possibility of the existence of local perched water tables should be borne in mind and the effects of seasonal flooding (where this occurs) should not be overlooked.

Category (3): In regions where the climate is dry throughout most of the year (annual rainfall 250 mm or less), the moisture content of the subgrade under an impermeable pavement will be low. For design purposes a value of 80 per cent of the optimum moisture content obtained in the British Standard (Light) Compaction Test, 2.5 kg rammer method, should be used.

The methods of estimating the subgrade moisture content for design outlined above are based on the assumption that the road pavement is virtually impermeable. Dense bitumen-bound materials, stabilised soils with only very fine cracks, and crushed stone or gravel with more than 15 per cent of material finer than the 75 micron sieve are themselves impermeable (permeability less than 10^{-7} metres per second) and therefore subgrades under road pavements incorporating these materials are unlikely to be influenced by water infiltrating directly from above. However, if water shed from the road surface or from elsewhere, is able to penetrate to the subgrade for any season, the subgrade may become much wetter, in such cases the strength of subgrades with moisture conditions in Category (1) and Category (2) should be assessed on the basis of saturated CBR samples as described in Section 3.2. Subgrades with moisture conditions in Category (3) are unlikely to wet up significantly and the subgrade moisture content for design in such situations can be taken as the optimum moisture content given by the British Standard (Light) Compaction Test 2.5 kg rammer method.

3.2 Determining the Subgrade Strength

Having estimated the subgrade moisture content for design, it is then possible to determine the appropriate design CBR value at the specified density. It is recommended that the top 250 mm of all subgrades should be compacted during construction to a relative density of at least 100 per cent of the maximum dry density achieved in the British Standard (Light) Compaction Test 2.5 kg rammer method, or at least 93 per cent of the maximum dry density achieved in the British Standard (Heavy) Compaction Test using the 4.5. kg rammer. With modern compaction plant a relative density of 95 per cent of the density obtained in the heavier compaction test should be achieved without difficulty but tighter control of the moisture content will be necessary. Compaction will not only improve the subgrade bearing strength but will reduce permeability and subsequent compaction by traffic.

As a first step it is necessary to determine the compaction properties of the subgrade soil by carrying out standard laboratory compaction tests. Samples of the subgrade soil at the design subgrade moisture content can then be compacted in CBR moulds to the specified density and tested to determine the CBR values.

With cohesionless sands, the rammer method tends to overestimate the optimum moisture content and underestimate the dry density achieved by normal field equipment. The vibrating hammer method is more appropriate for these materials.

If samples of cohesive soils are compacted at moisture contents equal to or greater than the optimum moisture content, they should be left sealed for 24 hours before being tested so that excess pore water pressures induced during compaction are dissipated.

Alternatively, a more complete picture of the relationship between density, moisture content and CBR for the subgrade soil can be obtained by measuring the CBR of samples compacted at several moisture contents and at least two levels of compaction. The design CBR is then obtained by interpolation. This method is preferable since it enables an estimate to be made of the subgrade CBR at different densities and allows the effects of different levels of compaction control on the structural design to be calculated. Figure 3 shows a typical dry density/moisture content/CBR relationship for a sandy-clay soil that was obtained by compacting samples at five moisture contents to three levels of compaction: British Standard (Heavy) Compaction, 4.5 kg rammer method, British Standard (Light) Compaction, 2.5 kg rammer method, and a intermediate level of compaction. By interpolation, a design subgrade CBR of about 15 per cent is obtained if a relative density of 100 percent of the maximum dry density obtained in the British Standard (Light) Compaction Test is specified and the subgrade moisture content was estimated to be 20 per cent.

If saturated subgrade conditions are anticipated, the compacted samples for the CBR test should be saturated by immersion in water for four days before being tested. In all other cases when CBR is determined by direct measurement, the CBR samples should not be immersed since this results in over design.

In areas where existing roads have been built on the same subgrade, direct measurements of the subgrade strengths can be made using a dynamic cone penetrometer (Appendix C).

Except for direct measurement of CBR under existing pavements, in situ CBR measurements of subgrade soils are not recommended because of the difficulty of ensuring that the moisture and density conditions at the time of test are representative of those expected under the completed pavement.

Which ever method is used to obtain the subgrade strength, each sample or each test will usually give different results and these can sometimes cover a considerable range. For design purposes it is important that the strength of the subgrade is not seriously underestimated for large areas of pavement or overestimated to such and extent that there is a risk of local failures. The best compromise for design purposes is to use the lower ten percentile value i.e. that value which is exceeded by 90 per cent of the readings. The simplest way to obtain this is to draw a cumulative frequency distribution of strength as shown in **Figure 4**. If the characteristics of the subgrade change significantly over sections of the route, different subgrade strength values for design should be calculated for each nominally uniform section.

The structural catalogue requires that the subgrade strength for design is assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength. The classes are defined in Table 3.1. For subgrades with CBR's less than 2, special treatment is required which is not covered in this Road Note.

TABLE 3.1

Subgrade strength classes

Class	Range (CBR%)
S1	2
S2	3-4
S3	5-7
S4	8-14
S5	15-29
S6	30

If equipment for carrying out laboratory compaction and CBR tests is not available, a less precise estimate of the minimum subgrade strength class can be obtained from **Table 3.2**. This Table shows the estimated minimum strength class for five types of subgrade soil for various depths of water table, assuming that the subgrade is compacted to not less than 95 per cent of the maximum dry density attainable in the British Standard (Light) Compaction Test, 2.5 kg rammer method. The Table is appropriate for subgrade moisture Categories (1) and (2) but can be used for Category (3) if conservative strength estimates are acceptable.

The design subgrade strength class together with the traffic class obtained in Chapter 2 are then used with catalogue of structures to determine the pavement layer thicknesses (chapter 10) .

TABLE 3.2

Estimated design subgrade strength class under sealed roads in the presence of a water table

Depth of water table* from formation level (metres)	Subgrade strength class				
	Non-plastic sand	Sandy clay PI=10	Sandy clay PI=20	Silty clay PI=30	Heavy clay PI>40
0.5	S4	S4	S2	S2	S1
1	S5	S4	S3	S2	S1
2	S5	S5	S4	S3	S2
3	S6	S5	S4	S3	S2

* The highest seasonal level attained by the water table should be used.

Note: 1. Since the strength classes given in **Table 3.2** are based on estimated minimum CBR values, wherever possible the CBR should be measured by laboratory testing at the appropriate moisture content.

2. **Table 3.2** is not applicable for silt, micaceous, organic or tropically weathered clays. Laboratory CBR tests should be undertaken for these soils.

5. DRAINAGE AND ROAD SHOULDERS

5.1 The Drainage System

One of the most important aspects of the design of a road is the provision made for protecting the road from surface water or ground water. If water is allowed to enter the structure of the road, the pavement will be weakened and it will be much more susceptible to damage by traffic. Water can enter the road as a result of rain penetrating the surface or as a result of the infiltration of ground water. The road surface must be constructed with a camber so that it sheds rainwater quickly and the top of the subgrade or improved subgrade must be raised above the level of the local water table to prevent it being soaked by ground water.

A good road drainage system, which is properly maintained, is vital to the successful operation of a road and the road designs described in this Note are based on the assumption that the side drains and culverts associated with the road are properly designed and function correctly.

Drainage within the pavement layers themselves is an essential element of structural design because the strength of the subgrade used for design purposes depends on the moisture content during the most likely adverse conditions. It is impossible to guarantee that road surfaces will remain waterproof throughout their lives, hence it is important to ensure that water is able to drain away quickly from within the pavement layers.

5.2 Pavement Cross-section

The width of the carriageway and the overall geometric design of the road are dealt with in Overseas Road Note No. 5 (Transport and Road Research Laboratory (1988)). For design traffic volumes in excess of about 1000 vehicles per day, carriageway widths of at least 7 metres should be used throughout and additional lanes will be needed when the capacity of a two-lane road is exceeded.

Shoulders are an essential element of the structural design of a road, providing lateral support for the pavement layers. They are especially important when unbound materials are used in the pavement and for this type of construction it is recommended that shoulders should be at least 2 metres wide. For bound roadbases, shoulder width can be reduced if required. If there is a large volume of non-motorised traffic, then shoulder width should be increased to a minimum of 3 metres. In order to exclude water from the road, the top of the shoulders should be impermeable and a surface dressing or other seal may need to be applied. Unsurfaced shoulders are not generally recommended because they often require considerable maintenance if satisfactory performance is to be guaranteed. Sealed shoulders also prevent the ingress of water at the edge of the pavement, which is an area particularly vulnerable to structural damage. Shoulders should be differentiated from the carriageway e.g. by the use of edge markings, different sized aggregate or different coloured aggregate.

Crossfall is needed on all roads in order to assist the shedding of water into the side drains. A suitable value for paved roads is about 3 per cent for the carriageway, with a slope of about 4-6 per cent for the shoulders. An increased crossfall for the carriageway e.g. 4 percent, is desirable if the quality of the final shaping of the road surface is likely to be low for any reason.

There is evidence that there are also benefits to be obtained by applying steeper crossfalls to layers at successive depths in the pavement. The top of the sub-base should have a crossfall of 3-4 per cent and the top of the subgrade should be 4-5 per cent. These crossfalls not only improve the drainage performance of the various layers, but also provide a slightly greater thickness of materials at the edge of the pavement where the structure is more vulnerable to damage. The design thickness should be that at the centre line of the pavement.

5.3 Drainage of Layers

Provided the crossfalls indicated above are adhered to and the bituminous surfacing and the shoulders are properly maintained, rainwater falling on the road will be shed harmlessly over the shoulders. When permeable roadbase materials are used, particular attention must be given to the drainage of this layer. Ideally, the roadbase and sub-base should extend right across the shoulders to the drainage ditches as shown in Fig. 6. Under no circumstances should the ‘trench’ type of cross-section be used in which the pavement layers are confined between continuous impervious shoulders.

If it is too costly to extend the roadbase and subbase material across the shoulder, drainage channels at 3m to 5m intervals should be cut through the shoulder to a depth of 50 mm below sub-base level. These channels should be back-filled with material of roadbase quality but which is more permeable than the roadbase itself and should be given a fall of 1 in 10 to the side ditch. Alternatively a continuous drainage layer of pervious material of 75mm to 100mm thickness can be laid uunder the shoulder such that the bottom of the drainage layer is at the level of the top of the sub-base. This latter is by far the better of the two alternatives.

In circumstances where the subgrade itself is permeable and can drain freely, it is preferable that vertical drainage is not impeded. If this can be done by ensuring that each layer of the pavement is more permeable than the layer above, then the additional drainage layer through the shoulders (layer No. 7 in **Fig. 6**) is not required.

5.4 Shoulder Materials

Although the ideal solution is to extend the roadbase and sub-base outwards to form the shoulders, when the roadbase material is non-plastic it may lack sufficient cohesion to withstand the abrasive action of traffic unless it is sealed with a surface dressing. In circumstances where extending the roadbase is not possible and the shoulder is not to be sealed, the shoulder material should be selected using the same principles as for a gravel-surfaced road or a sub-base to carry construction traffic. Thus the material should be strong enough to carry occasional vehicles and should be as cohesive as possible without being too weak when wet. The material will normally be of sub-base quality and the soaked CBR value at the specified density should exceed 30 per cent, except perhaps in arid areas where the binding action of plastic fines may become a more important criterion.

It is also very desirable if at least the outer edge of the shoulder is able to support the growth of grasses which help to bind the surface and prevent erosion. On rural roads where the shoulders rarely need to carry traffic, excellent shoulder performance can be obtained if the whole of the shoulder is grassed. In these circumstances it is necessary for this grass to be cut regularly to prevent the level of the shoulder building up above the level of the carriageway and thereby causing water to be retained at the carriageway-shoulder interface where it can penetrate the road structure and cause structural weakening.

6.0 UNBOUND PAVEMENT MATERIALS

This chapter gives guidance on the selection of unbound materials for use as roadbase, sub-base, capping and selected subgrade layers. The main categories with a brief summary of their characteristics are shown in **Table 6.1.**

6.1 Roadbase Materials

A wide range of materials can be used as unbound roadbases including quarried rock, crushed and screened, mechanically stabilised, modified or naturally occurring 'as dug' gravels. Their suitability for use depends primarily on the design traffic level of the pavement and climate but all roadbase materials must have a particle size distribution and particle shape which provide high mechanical stability and should contain sufficient fines (amount of material passing the 0.425 mm sieve) to produce a dense material when compacted. In circumstances where several types of roadbase are suitable, the final choice should take into account the expected level of future maintenance and the total costs over the expected life of the pavement. The use of locally available materials is encouraged, particularly at low traffic volumes (i.e. categories T1 and T2). Their use should be based on the results of performance studies and should incorporate any special design features which ensure their satisfactory performance. As a cautionary note, when considering the use of natural gravels a statistical approach should be applied in interpreting test results to ensure that their inherent variability is taken into account in the selection process.

For lightly trafficked roads the requirements set out below may be too stringent and in such cases reference should be made to specific case studies, preferably for roads under similar conditions.

TABLE 6.1

Properties of unbound materials

Code	Description	Summary of Specification
GB1,A	Fresh, crushed rock	Dense graded, unweathered crushed stone, non-plastic parent fines
GB1,B	Crushed rock, gravel or boulders	Dense grading, PI<6, soil or parent fines
GB2,A	Dry-bound macadam	Aggregate properties as for GB1,B (see text), PI < 6
GB2,B	Water-bound macadam	Aggregate properties as for GB1,B (see text), PI < 6
GB3	Natural coarsely graded granular material including processed and modified gravels	Dense grading, PI < 6 CBR after soaking > 80
GS	Natural gravel	CBR after soaking > 30
GC	Gravel of gravel-soil	Dense graded, CBR after soaking > 15

Note: 1. These specifications are sometimes modified according to site conditions, material type and principal use (see text).
2. GB= Granular roadbase, GS = Granular sub-base, GC = Granular capping layer.

6.1.1 Crushed Stone

Graded crushed stone (GB1, A and GB1, B): Two types of material are defined in this category. One is produced by crushing fresh, quarried rock (GB1,A) and may be an all-in-product, usually termed a 'Crusher -run', or alternatively the material may be separated by screening and recombined to produce a desired particle size distribution. The other is derived from crushing and screening natural granular material, rocks or boulders (GB1,B) and may contain a proportion of natural, fine aggregate. Typical grading limits for these materials are shown in **Table 6.2**. After crushing, the material should be angular in shape with a Flakiness Index (British Standard 812, Part 105 (1990)) of less than 35 per cent. If the amount of fine aggregate produced during the crushing operation is insufficient, non-plastic angular sand may be used to make up the deficiency. In constructing a crushed stone roadbase, the aim should be to achieve maximum impermeability compatible with good compaction and high stability under traffic.

To ensure that the materials are sufficiently durable, they should satisfy the criteria given in **Table 6.3**. These are a minimum Ten Per Cent Fines Value (TFV) (British Standard 812, Part 111 (1990)) and limits on the maximum loss in strength following a period of 24 hours of soaking in water. The likely moisture conditions in the pavement are taken into account in broad terms based on climate. Other simpler tests e.g.

TABLE 6.2

Grading limits for graded crushed stone roadbase materials (GB1, A; Gb1,B)

BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
	37.5 mm(1)	28 mm	20 mm
50	100	-	-
37.5	95-100	100	-
28	-	-	100
20	60-80	70-85	90-100
10	40-60	50-65	60-75
5	25-40	35-55	40-60
2.36	15-30	25-40	30-45
0.425	7-19	12-24	13-27
0.075(2)	5-12	5-12	5-12

Note: 1. Corresponds approximately to the UK specification for wet-mix macadam (Department of Transport, 1986)

2. For paver-laid materials a lower fines content may be accepted.

TABLE 6.3

Mechanical strength requirements for the aggregate fraction of crushed stone roadbases (GB1,A; GB1,B) as defined by the Ten Per Cent Fines Test

Climates	Typical annual rainfall (mm)	Minimum 10% fines values (kN)	Minimum ratio wet/dry test (%)
Moist tropical, wet tropical and seasonally wet tropical	>500	110	75
Arid and semi-arid	<500	110	60

the Aggregate Impact Test (British Standard 812, Part 112, 1990) may be used in quality control testing provided a relationship between the results of the chosen test and the TFV has been determined. Unique relationships do not exist between the results of the various tests but good correlations can be established for individual material types and these need to be determined locally.

When dealing with materials originating from the weathering of basic igneous rocks the recommendations in Section 6.1.2 should be used.

The fine fraction of a GB1, A material should be non-plastic. For GB1,B materials the maximum allowable PI is 6. When producing these materials, the percentage passing the 0.075 mm sieve should be chosen according to the grading and plasticity of the fines. For materials with non-plastic fines, the proportion passing the 0.075 mm sieve may approach 12 per cent. If the PI approaches the upper limit of 6 it is desirable that the fines content be restricted to the lower end of the range. To ensure this, a maximum Plasticity Product (PP) value of 45 is recommended where

PP = PI x (percentage passing the 0.075 mm sieve)

In order to meet these requirements it may be necessary to add a low proportion of hydrated lime or cement to alter the properties of the fines. Such materials are commonly referred to as modified materials. Further details are given in Chapter 7.

These materials may be dumped and spread by grader but it is preferable to use a paver to ensure that the completed surface is smooth with a tight finish. The material is usually kept wet during transport and laying to reduce the likelihood of particle segregation.

The in situ dry density of the placed material should be a minimum of 98 per cent of the maximum dry density obtained in the British Standard (Heavy) Compaction Test, 4.5 kg rammer, or the British Standard Vibrating Hammer Test (British Standard 1377, Part 4 (1990)). The compacted thickness of each layer should not exceed 200 mm.

When properly constructed, crushed stone roadbases will have CBR values well in excess of 100 per cent. In these circumstances there is no need to carry out CBR tests.

Dry-bound macadam (GB2, A): Dry-bound macadam is a traditional form of construction, formerly used extensively in the United Kingdom, and is comparable in performance with a graded crushed stone. It has been used successfully in the tropics and is particularly applicable in areas where water is scarce or expensive to obtain. It is also suitable where labour-intensive construction is an economic option. The materials consist of nominal single-sized crushed stone and non-plastic fine aggregate (passing the 5.0 mm sieve). The fine material should preferably be well graded and consist of crushed rock fines or natural, angular pit sand.

The dry-bound macadam process involves laying single-sized crushed stone of either 37.5 mm or 50 mm nominal size in a series of layers to achieve the design thickness. The compacted thickness to each layer should not exceed twice the nominal stone size. Each layer of coarse aggregate should be shaped and compacted and then the fine aggregate spread onto the surface and vibrated into the interstices to produce a dense layer. Any loose material remaining is brushed off and final compaction carried out, usually with a heavy smooth-wheeled roller. This sequence is then repeated until the design thickness is achieved. To aid the entry of the fines, the grading of the 37.5 mm nominal size stone should be towards the coarse end of the recommended range. Economy in the production process can be obtained if layers consisting of 50 mm nominal size stone and layers of 37.5 mm nominal size stone are both used to allow the required total thickness to be obtained more precisely and to make better overall use of the output from the crushing plant.

Water-bound macadam (GB2,B): Water-bound macadam is similar to dry-bound macadam. It consists of two components namely a relatively single-sized stone with a nominal maximum particle size of 50 mm or 37.5 mm and well graded fine aggregate which passes the 5.0 mm sieve. The coarse material is usually produced from quarrying fresh rock. The crushed stone is laid, shaped and compacted and then fines are added, rolled and washed into the surface to produce a dense material. Care is necessary in this operation to ensure that water sensitive plastic materials in the sub-base or subgrade do not become saturated. The compacted thickness of each layer should not exceed twice the maximum size of the stone. The fine material should preferably be non-plastic and consist of crushed rock fines or natural, angular pit sand.

Typical grading limits for the coarse fraction of GB2A or GB2B materials are given in **Table 6.4**. The grading of M2 and M4 correspond with nominal 50 mm and 37.5 mm single-sized roadstones (British Standard 63 (1987)) and are appropriate for use with mechanically crushed aggregate. M1 and M3 are broader specification. M1 has been used for hand-broken stone but if suitable screens are available, M2, M3 and M4 are preferred.

Aggregate hardness, durability, particle shape and in situ density should each conform to those given above for graded crushed stone.

TABLE 6.4

Typical coarse aggregate gradings for dry-bound (GB2,A) and water-bound macadam (GB2,B)

BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve			
	M1	M2[(1)]	M3	M4[(2)]
75	100	100	100	-
50	85-100	85-100	85-100	100
37.5	35-70	0-30	0-50	85-100
28	0-15	0-5	0-10	0-40
20	0-10	-	-	0-5

Note: 1. Corresponds to nominal 50 mm single-sized roadstone.

2. Corresponds to nominal 37.5 mm single-sized roadstone. To aid the entry of fines, to coarser end of this grading is preferred.

6.1.2 Naturally Occurring Granular Materials

Normal Requirements for Natural Gravel and Weathered Rocks (GB3): A wide range of materials including lateritic, calcareous and quartzitic gravels, river gravels and other transported gravels, or granular materials resulting from the weathering of rocks can be used successfully as roadbases. Table 6.5 contains three recommended particle size distributions for suitable materials corresponding to maximum nominal sizes of 37.5 mm, 20 mm and 10 mm. Only the two larger sizes should be considered for traffic in excess of 1.5 million equivalent standard axles. To ensure that the material has maximum mechanical stability, the particle size distribution should be approximately parallel with the grading envelope.

TABLE 6.5

Recommended particle size distributions for mechanically stable natural gravels and weathered rocks for use as roadbases (GB3)

BS test sieve	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
(mm)	37.5 mm	20 mm	10 mm
50	100	-	-
37.5	80-100	100	-
20	60-80	80-100	100
10	45-65	55-80	80-100
5	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 requirements consistently, screening and crushing of the larger sizes may be required. The fraction coarser than 10 mm should consist of more than 40 per cent of particle with angular, irregular or crushed faces. The mixing of materials from different sources may be warranted in order to achieve the required grading and surface finish. This may involve adding fine or coarse materials or combinations of the two.

All grading analysis should be done on materials that have been compacted. This is especially important if the aggregate fraction is susceptible to breakdown under compaction and in service. For materials whose stability decreases with breakdown, aggregate hardness criteria based on a minimum soaked Ten Per Cent Fines Value of 50 kN or a maximum soaked Modified Aggregate Impact Value of 40 may be specified (British Standard 812, Part 112 (1990)).

The fines of those materials should preferably be non-plastic but should normally never exceed a PI of 6. As an alternative to specifying PI, a Linear Shrinkage not exceeding 3 may be specified.

If the PI approaches the upper limit of 6 it is desirable that the fines content be restricted to the lower end of the range. To ensure this, a maximum PP of 60 is recommended or alternatively a maximum Plasticity Modulus (PM) of 90 where

PM = PI x (percentage passing the 0.425 mm sieve)

If difficulties are encountered in meeting the plasticity criteria consideration should be given to modifying the material by the addition of a low percentage of hydrated lime or cement.

When used as a roadbase, the material should be compacted to a density equal to or greater than 98 per cent of the maximum dry density achieved in the British Standard (Heavy) Compaction Test, 4.5 kg rammer. When compacted to this density in the laboratory, the material should have a minimum CBR of 80 per cent after four days immersion in water (British Standard 1377, Part 4 (1990)).

Arid and semi-arid areas : In low rainfall areas in the tropics, typically with a mean annual rainfall of less than 500 mm, and where evaporation is high, moisture conditions beneath a well sealed surface are unlikely to rise above the optimum moisture content determined in the British Standard (Heavy) Compaction Test. In such conditions, high strengths (CBR>80 per cent) are likely to develop even when natural gravels containing a substantial amount of plastic fines are used. In these situations, for the lowest traffic categories (T1, T2) the maximum allowable PI can be increased to 12 and the minimum soaked CBR criterion reduced to 60 per cent at the expected field density.

Materials of Basic Igneous Origin: Materials in this group are sometimes weathered and may release additional plastic fines during construction or in service. Problems are likely to worsen if water gains entry into the pavement and this can lead to rapid and premature failure. The state of decomposition also affects their long term durability when stabilised with lime or cement. The group includes common rocks such as basalts and dolerites but also covers a wider variety of rocks and granular materials derived from their weathering, transportation or other alteration (British Standards Institution (1975) and Weinert (1980)). Normal aggregate tests are often unable to identify unsuitable materials in this group. Even large, apparently sound particles may contain minerals that are decomposed and potentially expansive. The release of these minerals may lead to a consequent loss in bearing capacity. There are several methods of identifying unsound aggregates. These include petrographic analysis to detect secondary (clay) minerals, the use of various chemical soundness tests e.g. sodium or magnesium sulphate (British Standard 812 Part 121 (1990)), the use of dye adsorption tests (Sameshima and Black (1979)) or the use of a modified Texas Ball Mill Test (Sampson and Netterberg (1989)). Indicative limits based on these tests include (i) a maximum secondary mineral content of 20 per cent, (ii) a maximum loss of 12 or 20 per cent after 5 cycles in the sodium or magnesium sulphate tests respectively (iii) a Clay Index of less than 3 and (iv) a Durability Mill Index of less than 90. In most cases it is advisable to seek expert advice when considering their use, especially when new deposits are being evaluated. It is also important to subject the material to a range of tests since no specific method can consistently identify problem materials.

Materials of Marginal Quality: In many parts of the world as-dug gravels which do not normally meet the normal specifications for roadbases have been used successfully. They include lateritic, calcareous and volcanic gravels. In general their use should be confined to the lower traffic categories (i.e. T1 and T2) unless local studies have shown that they have performed successfully at higher levels. Successful use often depends on specific design and construction features. It is not possible to give general guidance on the use of all such materials and the reader is advised to consult the appropriate source reference (e.g. CIRIA (1988), Lionjanga et al (1987), Netterberg and Pinard (1991), Newill et al (1987) and Rolt et al (1987)).

The calcareous gravels, which include calceretes and marly limestones, deserve special mention, Typically, the plasticity requirements for these materials, all other things being equal, can be increased by up to 50 per cent above the normal requirements in the same climatic area without any detrimental effect on the performance of otherwise mechanically stable bases. Strict control of grading is also less important and deviation from a continuous grading is tolerable.

6.2 Sub-bases (GS)

The sub-base is an important load spreading layer in the completed pavement. It enables traffic stresses to be reduced to acceptable levels in the subgrade, it acts as a working platform for the construction of the upper pavement layers and it acts as a separation layer between subgrade and roadbase. Under special circumstances it may also act as a filter or as a drainage layer. In wet climatic conditions, the most stringent requirements are dictated by the need to support construction traffic and paving equipment. In these circumstances the sub-base material needs to be more tightly specified. In dry climatic conditions, in areas of good drainage, and where the road surface remains well sealed, unsaturated moisture conditions prevail and sub-base specifications may be relaxed. The selection of sub-base materials will therefore depend on the design function of the layer and the anticipated moisture regime, both in service and at construction.

6.2.1 Bearing Capacity

A minimum CBR of 30 per cent is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95 per cent of the maximum dry density achieved in the British Standard (Heavy) Compaction Test, 4.5 kg rammer. Under conditions of good drainage and when the water table is not near the ground surface (see Chapter 3) the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the British Standard (Light) Compaction Test, 2.5 kg rammer. In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state. Except in arid areas (Category (3) in Chapter 3), if the roadbase allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the roadbase in previous (see Section 3.1), saturation of the sub-base is likely. In these circumstances the bearing capacity should be determined on samples soaked in water for a period of four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field. In order to achieve the required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size distribution may be required. Materials which meet the recommendations of **Tables 6.6**. and **6.7** will usually be found to have adequate bearing capacity.

6.2.2 Use as a Construction Platform

In many circumstances the requirements of a sub-base are governed by its ability to support construction traffic without excessive deformation or ravelling. A high quality sub-base is therefore required where loading or climatic conditions during construction are severe. Suitable material should posses properties similar to those of a good surfacing material for unpaved roads. The material should be well graded and have a plasticity index at the lower end of the appropriate range for an ideal unpaved road wearing course under the prevailing climatic conditions. These considerations form the basis of the criteria given in **Tables 6.6** and **6.7.** Material meeting the requirements for severe condition will usually be of higher quality than the standard sub-base (GS). If materials to these requirements are unavailable, trafficking trials should be conducted to determine the performance of alternative materials under typical site conditions.

In the construction of low-volume roads, where cost savings at construction are particularly important, local experience is often invaluable and a wider range of materials may often be found to be acceptable.

TABLE 6.6

Recommended plasticity characteristics for granular sub-bases (GS)

Climate	Liquid Limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical	<35	<6	<3
Seasonally wet tropical	<45	<12	<6
Arid and semi-arid	<55	<20	<10

TABLE 6.7

Typical particle size distribution for sub-bases (GS)
which will meet strength requirements

BS Sieve size (mm)	Percentage by mass of total aggregate passing test sieve
50	100
37.5	80-100
20	60-100
5	30-100
1.18	17-75
0.3	9-50
0.075	5-25

6.2.3 Sub-base as a Filter or Separating Layer

This may be required to protect a drainage layer from blockage by a finer material or to prevent migration of fines and the mixing of two layers. The two functions are similar except that for use as a filter the material needs to be capable of allowing drainage to take place and therefore the amount of material passing the 0.075 mm sieve must be restricted.

The following criteria should be used to evaluate a sub-base as a separating or filter layer.

a) The ratio $\frac{\text{D15 (coarse layer)}}{\text{D85 (fine layer)}}$ should be less than 5

Where D15 is the sieve size through which 15 per cent by weight of the material passess and D85 is the sieve size through which 85 per cent passes.

b) The ratio $\frac{\text{D50 (coarse layer)}}{\text{D50 (fine layer)}}$ should be less than 25

For a filter to possess the required drainage characteristics a further requirement is :

c) The ratio $\frac{\text{D15(coarse layer)}}{\text{D15(fine layer)}}$ should be between 5 and 40

These criteria may be applied to the materials at both the roadbase/sub-base and the sub-base/subgrade interfaces. Further details can be obtained in the appropriate reference e.g. (NAASRA (1983)).

6.3 Selected Subgrade Materials and Capping Layers (GC)

These materials are often required to provide sufficient cover on weak subgrades. They are used in the lower pavement layers as a substitute for a thick sub-base to reduce costs. The requirements are less strict than for sub-bases. A minimum CBR of 15 per cent is specified at the highest anticipated moisture content measured on samples compacted in the laboratory at the specified field density. This density is usually specified as a minimum of 95 per cent of the maximum dry density in the British Standard (Heavy) Compaction Test, 4.5 kg rammer. In estimating the likely soil moisture conditions, the designer should take into account the functions of the overlying sub-base layer and its expected moisture condition and the moisture conditions in the subgrade. If either of these layers is likely to be saturated during the life of the road, then the selected layer should also be assessed in this state. Recommended gradings or plasticity criteria are not given for these materials. However, it is desirable to select reasonably homogeneous materials since overall pavement behaviour is often enhanced by this. The selection of materials which show the least change in bearing capacity from dry to wet is also beneficial.

10. STRUCTURE CATALOGUE

The basis of the catalogue has been described in section 1.7 and most of the information necessary to use it is contained in the main chapters of this Road Note. The cells of the catalogue are defined by ranges of traffic (Chapter 2) and subgrade strength (Chapter 3) and all the materials are described in Chapters 6 to 9. A summary of requirements and reference chapters relevant to each design chart is given in **Table 10.1.**

Although the thickness of layers should follow the design whenever possible, some limited substitution of materials between sub-base and selected fill is allowable based in the structural number principles outlined in the AASHTO guide for design of pavement structures (AASHTO (1986)). Where substitution is allowed, a note is included with the design chart.

The charts are designed so that, wherever possible, the thickness of each lift of material is obvious. Thus, all layers less than 200 mm will normally be constructed in one lift and all layers thicker than 300 mm will be constructed in two lifts. Occasionally layers are of intermediate thickness and the decision on lift thickness will depend on the construction plant available and the ease with which the density in the lower levels of the the lift can be achieved. The thickness of each lift need not necessarily be identical and it is often better to adjust the thickness according to the total thickness required and the maximum particle size by using a combination of gradings from **Table 6.2**.

In charts 3, 4 and 7 where a semi-structural surface is defined, it is important that the surfacing material should be flexible (Chapter 8) and the granular roadbase should be of the highest quality, preferably GB1, A. In traffic classes T6, T7 and T8 only granular roadbases of type GB1 or GB2 should be used, GB3 is acceptable in the lower traffic classes. For lime or cement-stabilised materials, the charts already define the layers for which the three categories of material may be used.

TABLE 10.1

Summary of material requirements for the design charts

Chart No	Surfacing	Roadbase	Refer to chapters
1	Double surface dressing	T1-T4 use GB1, GB2 or GB3 T5 use GB1,A or GB1,B T6 must be GB1,A	6 and 9
2	Double surface dressing	T1-T4 use GB1, GB2 or GB3 T5 use GB1 T6, T7, T8 use GB1A	6, 7 and 8
3	‘Flexible’ asphalt	T1-T4 use GB1 or GB2 T5 use GB1 T6 use GB1,A	6 and 8
4	‘Flexible’ asphalt	T1-T4 use GB1 or GB2 T5 use GB1 T6-T8 use GB1,A	6, 7 and 8
5	Wearing course and basecourse	GB1,A	6 and 8
6	Wearing course and basecourse	GB1 or GB2	6, 7 and 8
7	High quality single seal or double seal for T4. ‘Flexible’ asphalt for T5-T8	RB1, RB2 or RB3	8 and 9
8	Double surface dressing	CB1, CB2	7 and 9

The choice of chart will depend on a variety of factors but should be based on minimising total transport costs as discussed in Section 1.3 Factors that will need to be taken into account in a full evaluation include,

- the likely level and timing of maintenance
- the probable behaviour of the structure
- the experience and skill of the contractors and the availability of suitable plant.
- the cost of the different materials that might be used
- other risk factors

It is not possible to give detailed on these issues. The charts have been developed on the basis of reasonable assumptions concerning the first three of these, as described in the text, and therefore the initial choice should be based on the local costs of the feasible options. If any information is available concerning the likely behaviour of the structures under the local conditions, then a simple risk analysis can also be carried out to select the most appropriate structure (e.g. Ellis (1975)). With more detailed information, it should be possible to calibrate one of the road investment models such as HDM-III (Watanatada et at (1987)) or RTIM-2 (Parsley and Robinsor (1982) and then to use the model to calculate the whole life costs associated with each of the possible structures thereby allowing the optimum choice to be made. For many roads, especially those that are more lightly trafficked, local experience will dictate the most appropriate structures and sophisticated analysis will not be warranted.

EXAMPLE ON TRAFFIC DATA COLLECTION, TRAFFIC ANALYSIS AND PAVEMENT DESIGN

Traffic Analysis and Pavement Design have been made on the basis of ROAD NOTE-31 (TRL, U.K., 1993 version), reports on ROAD MATERIALS & STANDARD STUDY BANGLADESH (June, 1994), and literature from INDIAN ROAD CONGRESS (IRC).

Daily Traffic Counting :

Daily Traffic data on 7-day, 24 hour and both directions are to be counted. As per ROAD MATERIALS & STANDARD STUDY BANGLADESH (Vol.VIIB, Appendices, June-1994) , following points are to be remembered while collecting Traffic data :-

- The survey site should be such as to provide good visibility of traffic.
- Proper shelter/tents and safety to be provided to the stafff (specially in night survey and in remote area).
- The survey location should be atleast 1 km away from any large or small growth center to avoid large numbers of non-motorised vehicles. Also the site should be away from a Ferry ghat.
- Any seasonal variation in traffic flow due to agricultural production to be considered. Also several times in a year traffic data may be collected.

Data for each of the 7 consecutive days are to be collected using separate FORMAT. Then by summing up the 7-day data and dividing the summation by 7, the Average Annual Daily Traffic (AADT) in both directions, for each category of traffic for the year, are determined.

Converting Different Category of Daily Traffic to PCU (Passenger Car Unit) :

The result of the presence of slow moving vehicles in traffic stream is that it affects the free flow of traffic. The interference results in a fall in mean speeds. A way of accounting for the interaction of various kinds of traffic is to express the capacity of roads in terms of a common unit. The unit generally employed is the ‘Passenger Car Unit’ (PCU). The PCU Factors for conversion of different types of vehicles into equivalent passenger car units based on their relative interference value are given in the following Table from the Road Material & Standard Study Bangladesh (June-1994):-

Vehicle Type	*PCU Factor*
Truck	3.0
Bus	3.0
Minibus	3.0
Pickup	1.0
Car	1.0
Autorickshaw	0.75
Motorcycle	0.75
Bicycle	0.5
Rickshaw	2.0
Bullock-Cart	4.0

By multiplying the both way AADT (Average Annual Daily Traffic) for each category of traffic, by the respective PCU Factor and then summing up, the Total PCU may be obtained (shown in the worked out example). This is the Total No.of Traffic in terms of PCU per day, in both directions.

Checking the Road-Capacity or Width-Selection :

From total no. of Traffic in terms of PCU, fixing-up the road-width or determining the adequacy of the existing width, may be done using the guidelines of ROAD MATERIALS & STANDARD STUDY BANGLADESH (Vol. VIIA, June-1994).

Pavement width of Feeder Roads under LGED is 3.66m. According to the above report, the cross-section of currently used feeder roads of RHD comprises of 3.7 m wide pavement with 1.8 m soft unsurfaced shoulders. The nominal and ultimate capacities for these configurations are :--

- 583 PCU per hour , (6478 PCU per Day)
- 980 PCU per hour , (10,900 PCU per Day)

Cumulative Standard Axle Determination :

In the design method ROAD NOTE-31 (TRL, UK, 1993), the traffic is defined in terms of the cumulative number of standard axles (8160 Kg) to be carried during the design life of the road. It is well recognised that the structural damage caused by a vehicle depends on the axle load it imposes on the road, and the equivalent axle load concept is the best method available, for design purposes, to handle the large spectrum of axle loads actually applied to a pavement. For the purpose of structural design of road pavement, cars and similar sized vehicles can be ignored and only the total number and the axle loadings of the heavy vehicle that use the road during its design life need to be considered. In this context, heavy vehicles are defined as those having a unladen weight of 3000 Kg or more.

NOTE : Equivalence Factor = $\left(\frac{\text{Axle Load, in Kg}}{8160}\right)^{4.5}$, used for converting axle load of different vehicles to a common unit.

According to Axle Load Survey of Bangladesh (RMSS Report), only Truck, Bus and Minibus to be considered for road design purpose. On the basis of the analysis of existing data and further data collected by RMSS, the following recommendations are made for Equivalent Standard Axles (ESA) for Feeder Roads :--

ESA for Truck for All Feeder Roads of Bangladesh - 1.0 }
ESA for Bus for All Feeder Roads of Bangladesh - 0.5 } From RMSS, Vol IX B, Axle Load Survey Results
ESA for Minibus for All Feeder Roads of Bangladesh - 0.2 }

Growth Rate: An estimate of likely growth rate can be obtained by studying the past trends in traffic growth. According to Indian Road Congress (IRC-37-1984), if adequate data is not available, then an average value of 7.5 percent may be adopted for rural routes. However according to RMSS report, 8% growth rate per annum have been considered.

Design Life : It is considered appropriate that roads in rural areas should be designed for a life of 10 to 15 years but provision must be made in the design for progressive strengthening of the road. Arterial roads should normally be designed for 15 years life and others for 10 years. Urban roads may, however, be designed for a longer life based on judgement and depending on the rate of growth of the traffic expected. For LGED projects Design Life has been considered as 10 years for feeder roads in rural areas.

Cumulative Standard Axle for different category of Traffic (Trucks, Bus, Minibus)

This is calculated using the following formula for the design period and assumed annual growth rate, as shown below :-

$$CV = 365 \times AADT \times ESA \times \frac{(1 + r/100)^n - 1}{r/100}$$

$$C = \sum CV \text{ (for all heavy vehicle types)}$$

Where, CV	=	Cumulative Standard Axles, for each type of vehicle (Truck, Bus, Minibus), in both directions, in design life
AADT	=	Average Annual Daily Traffic for Vehicle Type at year of opening
ESA	=	Average Equivalent Standard Axle for Vehicle Type (from axle-load-survey of the particular country/area)
r	=	Growth Rate, in percentage
n	=	Design Period, in year
C	=	Cumulative Standard Axles for All Heavy Vehicle

NOTE : Total No. of Cumulative Standard Axles is expressed in Million Standard Axles or MSA (dividing the Total by 10^6).

Design Cumulative Standard Axle Determination :

For a Single-Lane road (3.66 m wide), traffic tends to be more channelised on single lane roads than on a two lane road.To allow this concentration of wheel load repetitions, the Design Cumulative Standard Axle should be based on the total number of commercial vehicles per day in both directions multiplied by 2 (two).

In case of a Double Lane road, the Design Cumulative Standard Axle is based on the total number of commercial vehicles per day in both directions.

DAILY TRAFFIC-COUNT DATA SHEET

(Sample Survey)

Road: **Kazihat--Rasulpur Rd, at Dhamrai-Thana, Dhaka-District** .

Sheet: 1 Date: **17th January, 1998** Day: **Saturday .**

NO. OF TRAFFIC IN BOTH DIRECTIONS (Counted On 24 Hours, Continuous Basis,For the Day)

Vehicle Type	*Time* 12:01am--8:00am	8:00am -- 4:00pm	4:00pm -- 12:00pm	*Total*
Truck	///	//// //// //// //// //// /	//// //// ///	42
Bus	/	//// //// ///	////	19
Minibus	///	//// //// //// //// //// / /// //// //// //// //	//// //// ////	65
Pickup	0	//	/	3
Car	0	////	///	8
Autorickshaw (including Tempo)	/	///	/	5
Motorcycle	///	//// //// ////	//// ////	27
Bicycle	//// //// ///	//// //// //// //// //// / /// //// //// //// //// //// / /// //// //// ///	//// //// //// //// //	108
Rickshaw	//// //// //// ////	//// //// //// //// //// / /// //// //// //// //// //// / /// //// //// //// //// //// / /// //// //// //// //// //// //// /	//// //// //// //// //// //// //// //// //// //// //// ////	199
Bullock-Car	//	//// ////	////	15

Comment\Observation on Weather Condition \ Seasonal Variation of Traffic \ Other Features :-------------

Enumerators :

12:01am to 8:00am--- Name: ______________________ Signature :
8:00 am to 4:00pm --- Name: ______________________ Signature :
4:00 pm to 12:00pm--- Name: ______________________ Signature :

Supervising Engr. at Field: Name: ______________________ Signature :

TRAFFIC ANALYSIS & FLEXIBLE PAVEMENT DESIGN FORMAT

Road / Location: **Kazihat to Rasulpur Road at Dhamrai Thana of Dhaka District**

Portion Considered: **Ch.122 m (Kazihat) to Ch. 7222m (Alipara) Portion for Reconstruction & Improvement**

Traffic Count Period: **17-1-98 (Sat) to 23-1-98 (Fri), on 24 hour Continuous Count Basis.**

Summerised Both Direction Traffic Data (using 7-day field data / one example enclosed) & **Road Capacity Determination** :

Vehicle Type	*Date & Day* Jan-17 SAT	Jan-18 SUN	Jan-19 MON	Jan-20 TUE	**Jan-21 WED**	Jan-22 THR	Jan-23 FRI	*7-day Total*	*Av. Annual Daily Traffic* in 1998	*PCU Factor*	*1998 PCU* (bothway & daily basis)
Truck	*42*	*37*	*43*	*77*	*180*	*60*	*49*	**488**	**70**	3.0	**210**
Bus	*19*	*21*	*17*	*23*	*26*	*18*	*17*	**141**	**20**	3.0	**60**
Minibus	*65*	*61*	*62*	*73*	*107*	*69*	*72*	**509**	**73**	3.0	**219**
Pickup	*3*	*3*	*4*	*6*	*14*	*2*	*2*	**34**	**5**	1.0	**5**
Car	*8*	*5*	*10*	*11*	*20*	*11*	*8*	**73**	**10**	1.0	**10**
Autorickshaw	*5*	*6*	*9*	*8*	*31*	*10*	*8*	**77**	**11**	0.75	**8.25**
Motorcycle	*27*	*31*	*28*	*35*	*98*	*36*	*27*	**282**	**40**	0.75	**30**
Bicycle	*108*	*113*	*105*	*143*	*333*	*120*	*94*	**1016**	**145**	0.5	**72.5**
Rickshaw	*199*	*224*	*228*	*280*	*677*	*243*	*217*	**2068**	**295**	2.0	**590**
Bullock-Cart	*15*	*14*	*17*	*31*	*58*	*22*	*19*	**176**	**25**	4.0	**100**

• PCU (Passenger Car Unit) Factors have taken from RMSS Report (June-1994). • **WED = Hat Day** Total PCU= . **1305**

So, for this traffic 3.66m wide Feeder road is : **Sufficient** .

NOTE : As per **ROAD MATERIALS & STANDARD STUDY BANGLADESH (Vol. VIIA, June-1994)** , for 3.66m wide RHD Feeder Rd with 1.8m unsurfaced shoulders, the NOMINAL & ULTIMATE CAPACITIES are respectively 6478 & 10,900 PCU per day.

Cumulative Traffic Analysis (bothway) :

Vehicle Type (Only Truck, Bus, Minibus)	Av. Annual Daily Traffic AADT, bothway	Equivalent Std. Axle (Feeder Rd) . ESA	Growth Rate . r	Design Life . n	Cumulative Standard Axle (each type vehicle) . $365 \times \text{AADT} \times \text{ESA} \times \frac{(1+r/100)^n - 1}{r/100}$
Truck	70	1.0	8 % Assumed	10 Yr Assumed	370132
Bus	20	0.5			52876
Minibus	73	0.2			77199
Other Vehicle	Negligible				Negligible

Equivalent Standard Axles are as per RMSS Recommendation (Vol IXB, Axle Load Survey Result, June-1994) for Feeder Roads. Accordingly only three types of vehicles are to be considered.

Total = **500207** or **0.50** Million (bothway ESA, in design life)

• **NOTE :** For Single-Lane Roads, traffic is more channelised than on two lane roads. To allow for this concentration of wheel load repetitions the design should be based on the Total No. of cumulative ESA in both directions multiplied by Two (according to ROADNOTE-31 & IRC).

- Therefore, for Single-Lane road, DESIGN CUMULATIVE STANDARD AXLE = **0.50** × **2** = **1.00** Million
  Hence, Traffic Class = **T_3** (as per ROAD NOTE-31, of TRL/UK, 1993)

- Subgrade CBR (found from Lab.Test) is = **3.2 %** , so Type = **S_2** (as per ROAD NOTE-31, of TRL/UK, 1993)

Traffic Class, in 10^6 ESA (ROADNOTE-31, TRL/1993)	T_1 = < 0.3 Million T_2 = 0.3 to 0.7 Million T_3 = 0.7 to 1.5 Million T_4 = 1.5 to 3.0 Million

Subgrade CBR Type (ROADNOTE-31, TRL/1993)	S_1 = 2 S_2 = 3, 4 S_3 = 5 to 7 S_4 = 8 to 14

Pavement Structure Recommended (from Chart-1 & 3 of RODE NOTE-31,TRL/UK,1993) for Granular Road Base & **T3 / S_2** Category are :--

175 mm Sub-base + 200 mm Base + Double Surface Dressing (Fig. 6-A)

or 175 mm Sub-base + 175 mm Base + 50 mm Carpeting (Fig. 6-B)

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APPENDIX-H

COMPOSITION OF THE TECHNICAL WORKING GROUP

Member Nominated by the Chief Engineer, LGED

1.	Mr. Rubaiyat Nurul Hasan Additional Chief Engineer, LGED	Convener
2.	Mr. Md. Abdul Karim PD, RRMIMP-2	Member
3.	Mr. Makbul Hossain PD, RDP-4, LGED	Member
4.	Mr. Md. Abdus Shaheed PD, RDP-18, LGED	Member
5.	Mr. Dick Lundberg Sr. Adviser Institutional Support, ISP, LGED	Member
6.	Mr. Abdur Rahman Khan Team Leader, Technical Audit Services, RRMIMP-2, LGED	Member
7.	Mr. Md. Moksed Alam Road Infrastructure Design Consultant, ISP, LGED	Member
8.	Mr. A.N.M. Wahiduddin Laboratory Consultant, ISP, LGED	Member
9.	Mr. Md. Mizanur Rahman Road Infrastructure Design Specialist, ISP, LGED	Member

Members Co-opted by the Technical Working Group, LGED

1. Mr. H. R. Akanda
   Management Consultant, ISP, LGED

2. Mr. Saroj Kumar Sarker
   Superintending Engineer (Design), LGED

3. Mr. Md. Nurul Islam
   PD, SSWRDSP, LGED

4. Mr. F. J. Jung
   Chief Technical Adviser, RDP-14, LGED

5. Mr. Mukhlesur Rahman
   DTL, RRMIMP-2, LGED

6. Mr. Taiyeb Ali
   Team Leader, RDP-3, LGED

7. Mr. Md. Abdul Mannan
   Pavement Engineer, RDP-18, LGED

TERMS OF REFERENCE (TOR) OF THE TECHNICAL WORKING GROUP

The TOR of the Technical Working Group were :-

1. Co-ordinate road construction trial activities under different projects of LGED.

2. Review technical specification and design standard of Feeder Road Type-B (FRB) and Rural Roads covering sub-grade, sub-base, base-course, surface etc.

3. Consider different types of road surfacing, such as, Otta Surfacing, Double Bitumen Surface Treatment (DBST), Bituminous Carpet (BC) and other types of surfacing.

4. Formulate Road Pavement Design Manual of LGED

5. Any other relevant issue.