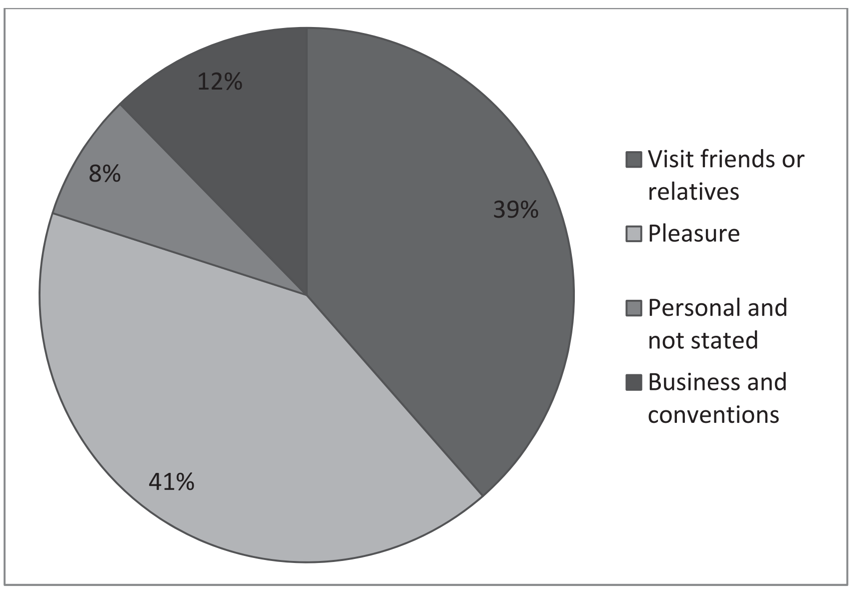
1. Introduction to Transport Planning and Engineering
CHAPTER ONE

1.1 The Role of Transport

Transport systems in Canada provide for the movement of both people and goods. The total transportation infrastructure includes road, rail, air, pedestrian, pipeline and marine transportation and plays a vital role in the Canadian economy. Except for the two last modes, the others make use of pavements or some other sort of load supporting structure. For example, railways operate on rails, ties and ballast, not dissimilar in concept to a road or airfield pavement structure [TAC 97]. From a broad perspective, Canada's transportation system plays a vital role in a country that encompasses nearly 10 million square km but with a population only one tenth that of the United States.

Figure 1.1 shows the main reasons why Canadians travel and the diagram shows that most inter-city travel (trips > 80 km) is for vacation, visiting friends and family with business travel accounting only for a small share of the travel in 2004.

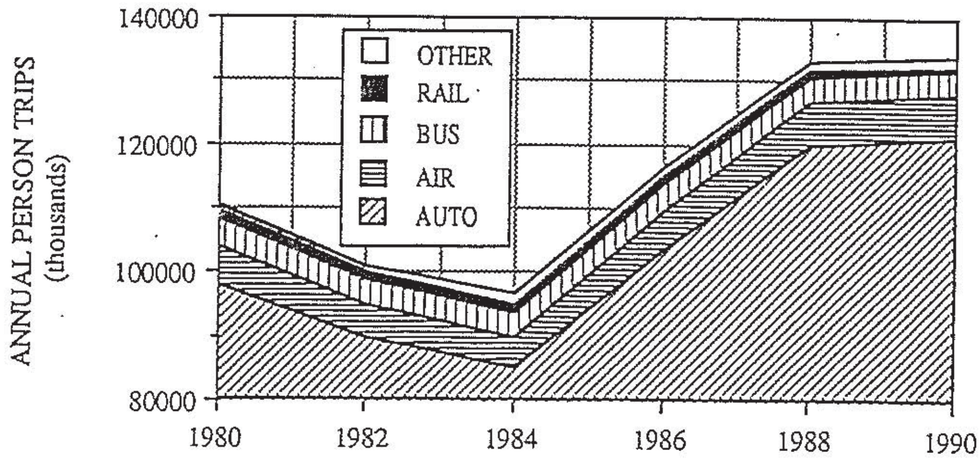


Adapted from: Source: Statistics Canada. Canadian Travel Survey. Domestic Travel, 2004

Figure 1.1 Domestic Inter-City Travel by Main Trip Purpose, 2004

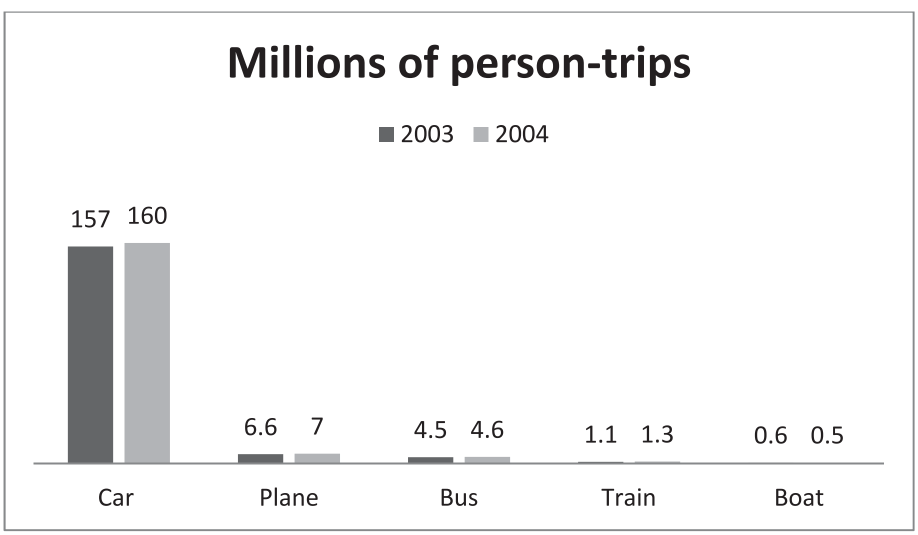
Figure 1.2 shows the trends in the number of inter-city trips per year in Canada in 2003 and 2004 and Figure 1.3 shows these numbers for 2003 and 2004. These two figures show clearly that travel has been dominated by car travel. More than 60 percent of inter-city travel is for trips of less than 150 km and the car captures about 90 percent of travel with this trip length. The total length of roads which support this car travel is approximately 840,000 km of which 63% is earth and gravel and 37% is paved or surface treated. However, the paved portion is the most intensely used with about 55% of all vehicle-km of travel occuring in the urban areas even though these areas only have about 15% of the total km [RTAC 92]. Figures 1.2 and 1.3 also show that air travel is the major public transport mode followed by inter-city bus and with rail travel capturing a very small share of inter-city person travel.

An important feature illustrated by Figure 1.2 is the downturn in travel between 1980 and 1984 which reflected the severe reduction in economic growth during this period. The 1992 survey will illustrate a similar reduction in inter-city travel because of the economic recession. For example, domestic air travel decreased by about 9 percent in 1991 and by a further 10 percent in 1992. Figure 1.3 shows that the inter-city travels continued to increase significantly after 1990 with about 30% in fourteen years. The car continues however to be the major transport mode for this category of travels.



(Source: Statistics Canada. Touriscope: Domestic Travel 1990. October, 1991)

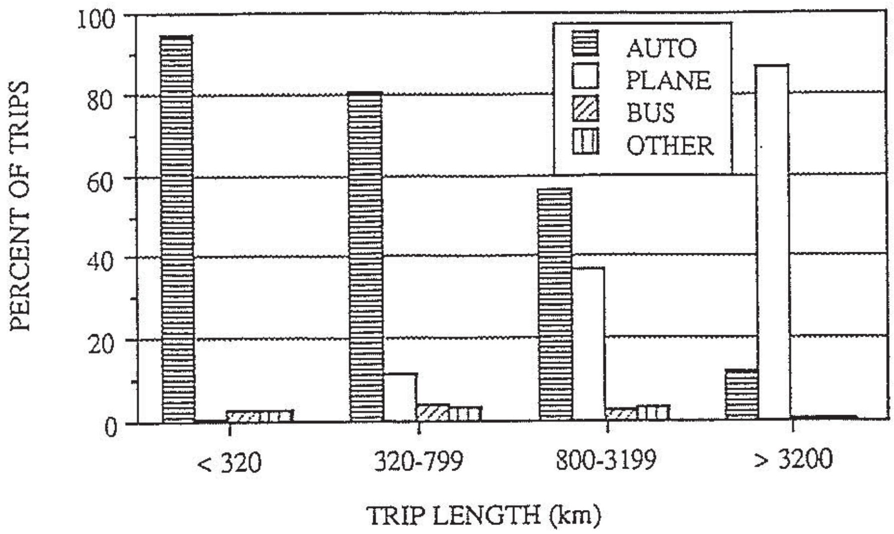
Figure 1.2 Trends in Annual Inter-City Person Trips in Canada 1980- 1990



(Adapted from: Source: Statistics Canada. Canadian Travel Survey. Domestic Travel, 2004)

Figure 1.3 Annual Inter-City Person Trips in Canada in 2003 and 2004

Figure 1.4 illustrates the choice of transport mode by Canadians as the trip length increased. The diagram illustrates very clearly that the automobile dominates for trips up to about 800 km while the share of travel captured by the air mode increases sharply for the longer trip lengths. For example, for trip lengths greater than 3,200 km the air mode captured about 80 percent of inter-city passenger travel in 1990. This increasing share of travel by air reflects the time budget constraints of most travelers and the higher costs of travel by car (trip plus accommodation costs) than by air.



Source: Statistics Canada. Touriscope: Domestic Travel 1990. October, 1991.

Figure 1.4 Modal Transport Share of Domestic Inter-City Travel as Trip Length Increases, 1990

The main economic role of freight transport is to provide for the movement between the points of production and consumption. Each unit of economic activity may be thought of in the manner illustrated in Figure 1.5. A unit receives various types of goods and services as inputs and dispatches other types of goods as outputs. A manufacturing plant receives inputs of raw materials and semi-finished products and dispatches semi-finished and finished products to other plants, warehouses and retail outlets. Freight terminals receive goods that are either consolidated into larger consignments for locations external to the urban area it serves, or goods that must be separated into smaller consignments for distribution within the urban area.

Each commercial activity will tend to locate at a position which helps it to maximize its profit and for many companies this means attempting to minimize the total costs of transporting both inputs and outputs. For example, paper companies use large volumes of relatively low value wood products and produce relatively high value outputs. Transport costs tend to dictate paper plant locations close to the forest product sources with proximity to final markets being less important. On the other hand, high technology companies require very low volumes of input materials and produce outputs with very high unit values are relatively unconstrained in their location decisions, with the major factors usually being access to a well-trained labour force and a high-quality living environment. The location of other companies, such as beverage companies, will be dictated mainly by access to the markets for their products because the products are dense and low value and transport costs are a major component of production costs. While transport services have an impact on the location decisions of companies this impact is much less than it used to be as transport costs have become cheaper relative to other production inputs.

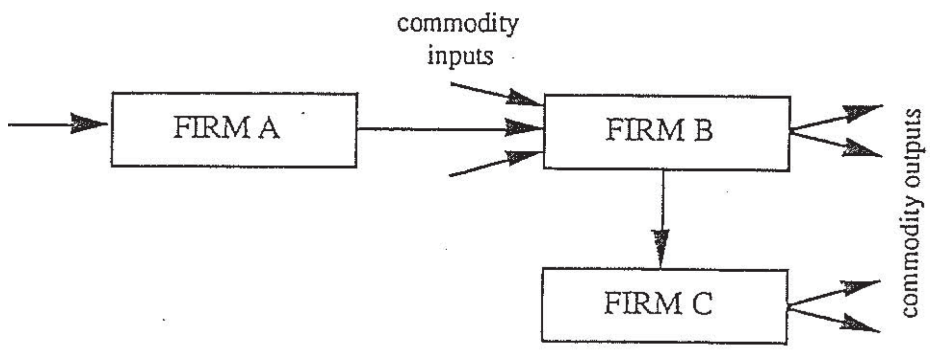


Figure 1.5 Interaction between Economic Activities and Transport Demands

1.2 The Transport System

Table 1.1 illustrates the components of the inter-city transport system in Canada. It must be recognized that the inter-city transport system is closely connected to the urban transport systems of Canada. Inter-city passenger trips and freight movements (except for natural resource movements) usually involve a significant urban component. Transport planning and engineering is concerned with supplying transport services and facilities that satisfy the demands for these services. Civil engineers normally supply these services in association with others. For example, civil engineers may plan, design and manage airports but they are normally not involved in operating the airline companies that use an airport. A similar situation exists for the highway transportation system, although highway authorities do regulate the characteristics of the vehicles that may use highway facilities. The underlined components of the table indicate those sub-systems for which civil engineers have been traditionally responsible.

Table 1.1 Components of the Transportation System

Mode	Vehicles	Infrastructure		
		Terminals	Links	Traffic Control
Roads	Cars & Trucks	Parking & freight terminals	Roads & bridges	Road signs, signals & IHVS*
	Buses	Bus terminals		
Air	Airplanes	Airports (incl. runways)	Air navigation systems	Air traffic control
Rail	Trains	Stations & freight yards	Railway tracks	Dispatch & signal systems

Water	Ferries & Ships	Ferry terminals & wharves	Waterways & canals	Vessel traffic services

* IVHS = intelligent highway & vehicle systems

Source: Royal Commission on National Passenger Transportation. "Directions - The Final Report of the Royal Commission on National Passenger Transportation. Volume 1, 1992.

Railway systems are different from the other transport modes in that rail companies normally own and operate the entire system of terminals, track and rolling stock. With the air, road and water modes the link infrastructure is normally provided by the public sector and the costs of supplying these facilities are normally recovered through a combination of user charges and general taxation revenues.

One of the major changes in transport systems over the past 25 years has been their continued market specialization and the integration between modes. The principal freight markets served by the modes in Canada are:

- **Highway System:** most manufactured goods and bulk commodities with trip lengths up to 1500-2000 km are moved by truck and most passenger transport for trips of up to 1000 km are by private car.

- **Air Transport System:** air services transport significant volumes of high value commodities and most inter-city passenger travel for trips greater than about 2000 km.

- **Railway System:** most bulk resources (grain, coal, lumber, etc.) are moved by train along with some inter-modal (container and trailer) traffic.

The broad process of transport planning is illustrated in Figure 1.6. The set of activities on the left-hand side are concerned with estimating the demands for particular transport modes. The diagram emphasizes that transport demand is a derived demand in that demands are determined by the spatial distributions of population and economic activities. The amount of travel and the mode of travel chosen depend on factors such as the relative prices of travel, times of travel, frequencies of travel services provided, and so on.

A first step in the process of transport planning and engineering is to estimate the future demand for transport services and the procedure used to construct transport demand models. The prices, times and frequency characteristics of the various modes of travel depend on the technology of transport (vehicles and link infrastructure) and the input factor costs (wages, energy costs, link infrastructure construction costs). Figure 1.6 indicates that the prices and levels of service offered by each mode interact with the forces generating transport demand and this interaction results in the transport flows on each of the modes.

A century ago most inter-city passenger and freight travel was by rail and in some cases by water. The two dominant modes at present, highway and air, only developed during the 20th century.

Vehicle technology (including engines, vehicle structures, navigation systems, etc.) developed along with the appropriate civil engineering technology to reduce transport costs and improve the quality of service through reduced travel times. For example, the wide body jet aircraft introduced in 1970 reduced the per-passenger costs substantially (because of lower labour costs and reduced fuel consumption per passenger) and this produced sharp increases in air travel during the 1970's and 1980's, particularly for non-business purposes. The IHVS (intelligent highway-vehicle system) technology is just beginning to emerge and it is expected that this will create tremendous opportunities for improving the quality and efficiency of road transport.

Figure 1.6 suggests that the transport modes also produce significant environmental impacts (noise, air pollution) and have significant resource consumption implications (70 percent of liquid petroleum products) which must be considered during their planning and design. The shaded boxes in Figure 1.6 emphasize the activities that are central to transport system planning and design decisions and which are the primary responsibility of the civil engineer.

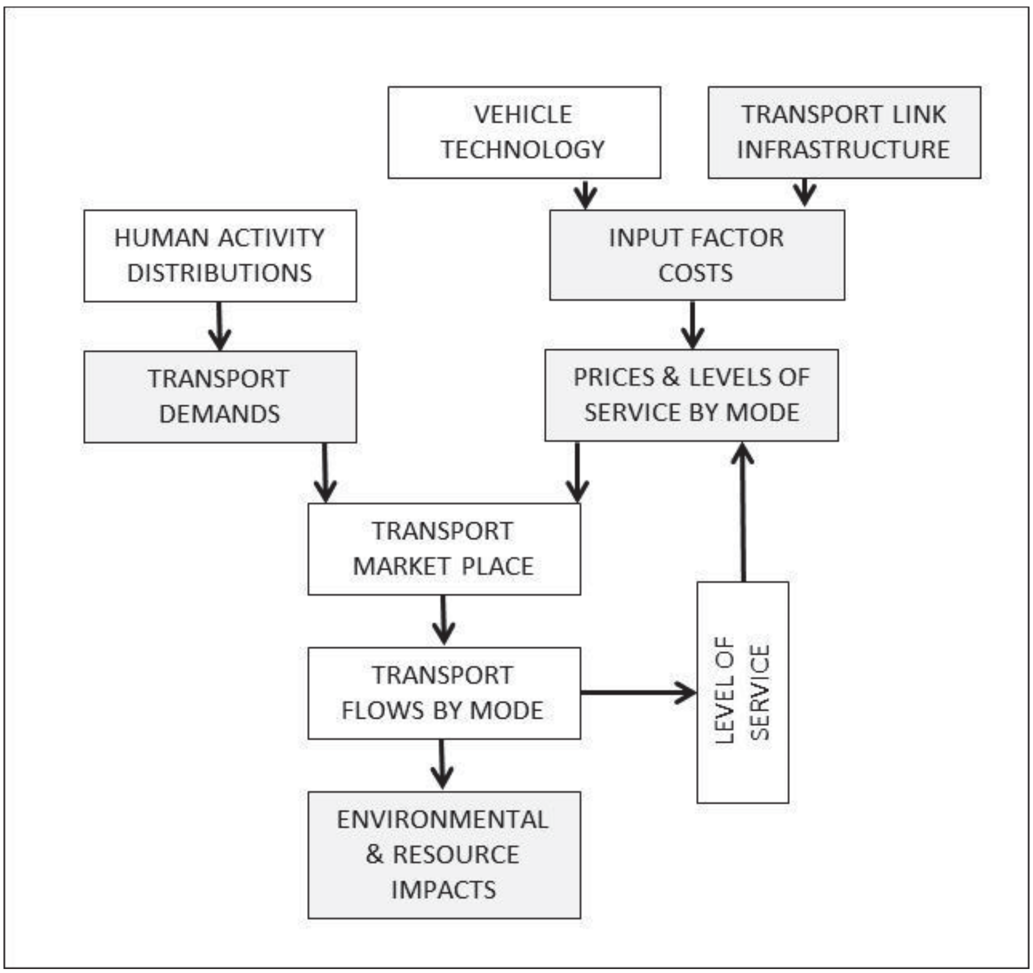


Figure 1.6 The Broad Process of Transport Planning

1.3 Transport Planning and Engineering Process

Figure 1.6 shows the steps involved in the process of transport planning and engineering, where the amount of detail generated increases as one proceeds through the process of planning and design. The diagram suggests that demand estimation is followed by a process of conceptual planning. The goal of conceptual planning is normally to generate one or more transport concepts that appear to be physically and economically feasible. Examples are the development of an initial set of highway location alternatives and the initial concept for a resource movement railway.

The next step of the process involves functional planning where the objective is to develop one or more of the concepts in more detail. This step allows the engineering problems to be approached in more detail and the costs to be estimated more accurately with an example being the functional planning of a section of a highway.

The third step of the process illustrated in Figure 1.7 is preliminary design. This step involves the more detailed technical design of the facilities which are components of a system. Examples are the detailed geometry of a rail line or of a highway, the preliminary design of bridges and pavements, and so on.

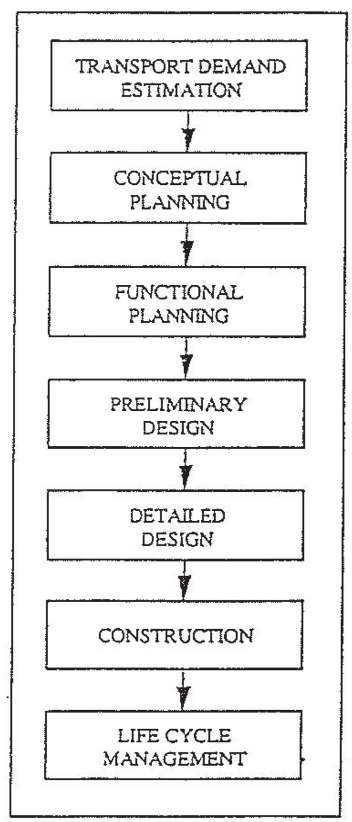


Figure 1.7 Transport Planning and Engineering Process

The fourth step identified in Figure 1.7 is detailed design, where the level of detail required is that necessary to prepare contract drawings. The objective of this phase is to develop the design to the point where a “third-party” is able to construct the project, or a component of the project, from the drawings. This involves the detailed design of structures, the selection of detailed specifications for materials, etc.

The final step identified in Figure 1.7 has been labelled life cycle management. This activity has been included to convey the idea that transport planning and engineering is an ongoing operation that does not end with construction. Facilities must be monitored, maintained and re-built and construction materials re-cycled. This last activity is becoming increasingly important, since massive investments were made in urban and highway facilities about 25 years ago, and this infrastructure is beginning to deteriorate.

The current value of the civil transportation infrastructure in the United States and Canada is well over twenty trillion dollars. Good management, as well as timely and cost-effective renovation, repair and rehabilitation technologies are essential to the preservation of these assets.

References:

[TAC 13] Transportation Association of Canada, Pavement Asset Design and Management Guide, Ottawa, 2013.

[TAC 97] Transportation Association of Canada, Pavement Design and Management Guide, Ottawa, 1997.

[RTAC 92] Roads and Transportation Association of Canada, “Transportation in Canada 1992 Report”, TAC Ottawa 1992.

2. Airport Planning and Design
CHAPTER TWO

2.1 Introduction

Airports have traditionally been regarded as public utilities to be operated and financially supported by governments. However, in the past decade, there has been a global trend toward making airports financially self-sufficient through the introduction of commercial goals and in some cases private ownership [Humphreys 00]. In terms of airport operations, this has meant more emphasis on efficient revenue generation from commercial sources. This change has been driven by the desire of public authorities to reduce the cost burden of their airports and to maximize revenue-earning potential.

This change in the management and fundamental operation has greatly affected the Canadian airport community. This "privatization" has various implications in terms of both operation and management of the airport. In terms of engineering, many of the airports may not be able to retain their own specialist engineering staff [CAPTG 00]. In effect, many airports will have to rely more on the private sector for knowledge, expertise, and a range of airfield pavement technologies. Such examples will include the need for quantifying pavement performance, life-cycle cost analysis, value engineering, and the implementation of pavement management systems so that all activities are carried out in the most cost-effective manner [Tighe 00]. The airport is a complex and dynamic organization, which consists of many interacting parts. Performance measures have a wide range, including airport capacity, safety, noise impact, security, accessibility, average flight delay and pavement condition [Humphreys 00, Braaksma 00].

2.2 Canadian Airports

In Canada, the trend toward commercialization has also been prevalent. With the introduction of the National Airport Policy (NAP) in 1994, Transport Canada transferred the management, operation and maintenance, and, in some cases, the ownership of airports to Canadian Airport Authorities (CAAs) or Local Airport Authorities (LAAs) as shown in Table 2.1.

A complete list of all the airports transferred to LAAs and CAAs, as well as more information related to the NAP is available on the Transport Canada website at https://www.tc.gc.ca/eng/programs/airports-policy-nas-1129.htm.

There are 726 certified airports in Canada, which are each categorized as one of: the National Airport system (NAS), regional and local airports, small airports, remote airports, and arctic airports. Within each of the five aforementioned categories, Transport Canada has a varying degree of involvement [TC 10].

Table 2.1 Airports Transferred from Transport Canada to CAAs or LAAs

Airport Category	Scheduled Passenger Traffic	Yearly Passengers	Unique Feature	Airports Transferred	Airports to be Transferred
National Airport System	Yes	>200,000	Handles 94% of air traffic in Canada	26	0
Regional/Local	Yes	<200,000	Eligible for Airports Capital Assistance Program (ACAP)	64	7
Small	No	n/a	Mostly recreational flights	30	1
Arctic	No	n/a	To become assigned as part of NAS or Remote Category	8	0
Remote	No	n/a	Provides year-round transportation to isolated communities	0	0

The federal government, through Transport Canada, was not ideally suited to function as an operator, financier, landlord, regulator and advisor [TC 10]. Consequently, national and regional imbalances were developed with regard to facilities and funding. In light of this and a growing global trend toward commercialization of airports [Humphreys 00], the Canadian NAP was developed to ensure the Canadian National Airport system is safe, commercially oriented, and cost-effective.

Under the NAP, the federal government maintains its role as regulator but changes its role from airport owner and operator to that of owner and landlord. Effectively this means that [TC 10]:

1. The 26 airports that are part of the National airport system (NAS) are owned by the federal government but are leased to the LAAs/CAAs. These authorities are responsible for financial and operational management. The NAS airports include those in the national and provincial and territorial capitals and airports that handle at least 200,000 passengers each year. Toronto's Lester B. Pearson International Airport (LBPIA) is an example of an NAS.

2. Ownership of regional/local and other smaller airports was transferred to regional interests (fewer than 200,000 passengers per year). Region of Waterloo International Airport is an example of a regional/local airport.

3. The small airports which the federal government owned that do not have scheduled passenger service (most of these are used for recreational flying), have been transferred to local interests through appropriate government processes. Niagara District Airport in Ontario is an example of a small airport.

4. Remote airports which provide exclusive reliable year-round access to isolated communities and which receive federal assistance have continued to be supported. Moosonee Airport in Ontario is an example of a small airport.

5. Federal arctic airports continue to be offered to the respective government for operation under the existing airport transfer programs. Watson Lake Airport in the Northwest Territories is an example of an arctic airport.

Overall, the federal government has continued to set safety and security standards for all Canadian airports by policy setting, airport transfer agreements, airport certification and regulation. However, the NAP has shifted the responsibility of operating Canada's airports to Local Airport Authorities (LAAs) [TC 10].

This transfer to LAAs started in the early 1990s. In 1993, Transport Canada transferred Vancouver International Airport, Edmonton International Airport, Calgary Airport and Dorval and Mirabel International Airports (Aéroports de Montreal) to LAAs [Braaksma 00]. In 1996, the Greater Toronto Airports Authority (GTAA) assumed responsibility for the management, operation and maintenance of LBPIA [GTAA 16] and some more recent transfers include Prince George Airport Authority in 2003.

2.3 Airport Planning and Design

Figure 2.1 illustrates the principal components of an airport system, which serves as a transfer point between the ground transport and air transport modes. Airport systems contain many components, and each must be carefully arranged and dimensioned to ensure efficient operation of the system.

For example, if the air terminal and ground transport systems are overloaded and congested, then many of the advantages of air transport are lost, particularly on short haul flights. The main focus of this chapter is with the design of the airside of airports.

Important inputs to the planning and design of airport systems are estimates of the future air travel demands to be handled by an airport. These demands include passenger volumes, freight volumes, aircraft movements, and ground transport demands. The demand between two destinations is a function of thousands of factors, including their respective populations, business communities, and attractiveness, as well as the transportation modes available between the two cities.

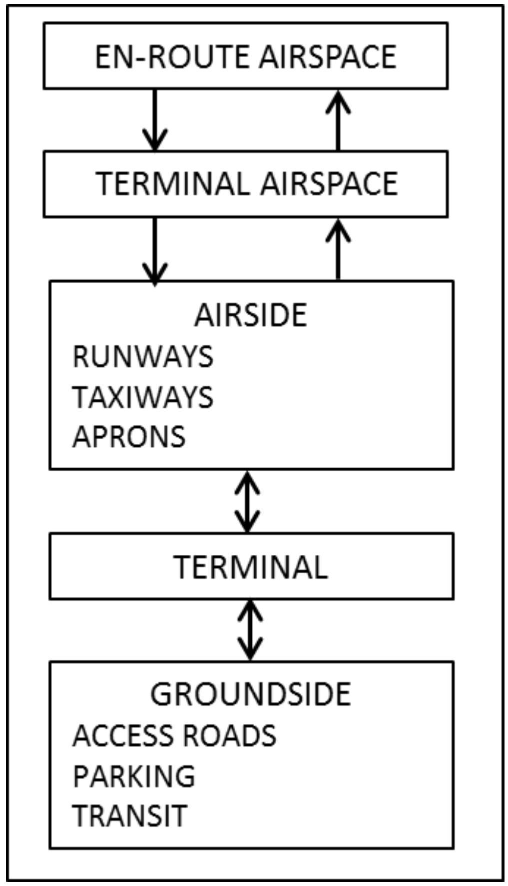


Figure 2.1 Components of the Air Transport System

In addition to the total annual number of passengers expected to pass through an airport, the airport planner must also know something about the route structure of the aircraft using an airport (e.g., hub and spoke versus mainly aircraft in transit), the aircraft types expected to use an airport (e.g., B747 versus DC9), and the proportions of transfers between flights that occur at an airport. Understanding the route structure along with the composition of aircraft types allows the airport planner to estimate the peaking characteristics of the demand (for passengers and aircraft movements) which are essential for establishing runway and terminal capacities.

In Canada, major airports are designed to handle 90 percent of the expected hourly volumes in the design year without significant delay with the remaining 10 percent representing the few very high hourly volumes that occur on major holidays (July, August, Easter, Christmas). The mix of aircraft types using an airport determines the runway lengths, taxiway exit locations, geometry of the airside facilities, gate configurations and spacing, turning movement requirements, etc.

2.3.1. Air Traffic Control

The main goals of Air Traffic Control (ATC) are to prevent conflicts between aircrafts, route aircraft through adverse weather, manage traffic to reduce congestion at key airports, and aid pilots in precise navigation required on approaches to runways. Each nation is responsible for their own airspace with control over oceans divided among nations. Usually when over land aircrafts will use ground-based navigation systems consisting of very high frequency Omni range (VOR) radios that send out directional signals. Aircrafts have devices with position indicators that show them where they are relative to given radial points. Over water, long range navigation systems are used.

2.3.2. Airspace Organization

New airports cannot be constructed, nor can the operations of existing airports be expanded significantly, without the airspace structure being properly understood. There are two basic types of flight rules for air traffic, known as visual flight rules (VFR) and instrument flight rules (IFR).

Table 2.2 VFR vs. IFR

Visual Flight Rules (VFR)	Instrument Flight Rules (IFR)

With IFR the safe separation between aircraft is maintained by ATC staff. In the most heavily travelled corridors only IFR are allowed. Air space throughout the world is carefully controlled with commercial aircraft using a complex system of airways and more-tightly controlled terminal control areas near large airports. Flights along the airways in Canada are controlled by a system of area control centres (i.e., Gander, Moncton, Montreal, Toronto, Winnipeg, Edmonton, and Vancouver) and movements around the two major Canadian airports by terminal control units (i.e., Toronto and Montreal).

ATC has two primary objectives, and these are safety and efficiency of aircraft movement. Controlled airspace is between 5,000 and 25,000 feet (1524m and 7620m, respectively) and aircraft are not allowed to fly in the controlled airspace unless they have first obtained clearance from ATC. The aircraft navigational equipment and pilot qualifications are of a prescribed standard and continuous communication is maintained between ATC units and aircraft.

The en-route airways in Canada mostly consist of a system of low level and high-level airways which connect points equipped with ground-based navigation aids. Low level airways extend from 2,200 feet (671m) above ground level (AGL) up to and not including 18,000 feet above sea level (ASL) and the low-level airways are approximately 4 nautical miles (nm) wide (*1 nautical mile ≈ 1.15 miles*). The high-level airways extend from 18,000 feet ASL to 45,000 feet ASL with aircraft being separated by 1,000 feet vertically.

In addition, westbound (180^o-359^o) aircraft fly at the even 1,000 feet altitudes while the eastbound (000^O-179^o) aircraft fly at the odd numbered 1,000 feet altitudes. There is no specific width to the high-level airways, but ATC establishes the lateral separation of aircraft. Figure 2.2 illustrates the typical arrival and departure paths for the simultaneous operation parallel runways 06-24L and 06-24R at LBPIA.

Toronto's airspace consists of a system of low and high-level airways connected to the terminal airspace system. En-route controllers control the movement of aircraft along the airways and when aircrafts reach the terminal airspace their control is handed-off to the terminal controllers. The Toronto terminal airspace extends out to a distance of 22 nm. Figure 2.2 shows the system of low and high-level airways and illustrates there are four entry points to the terminal airspace. The four VORs are located at Simcoe, Waterloo, Mans, and 26 nm south of LBPIA over Lake Ontario (LINGG).

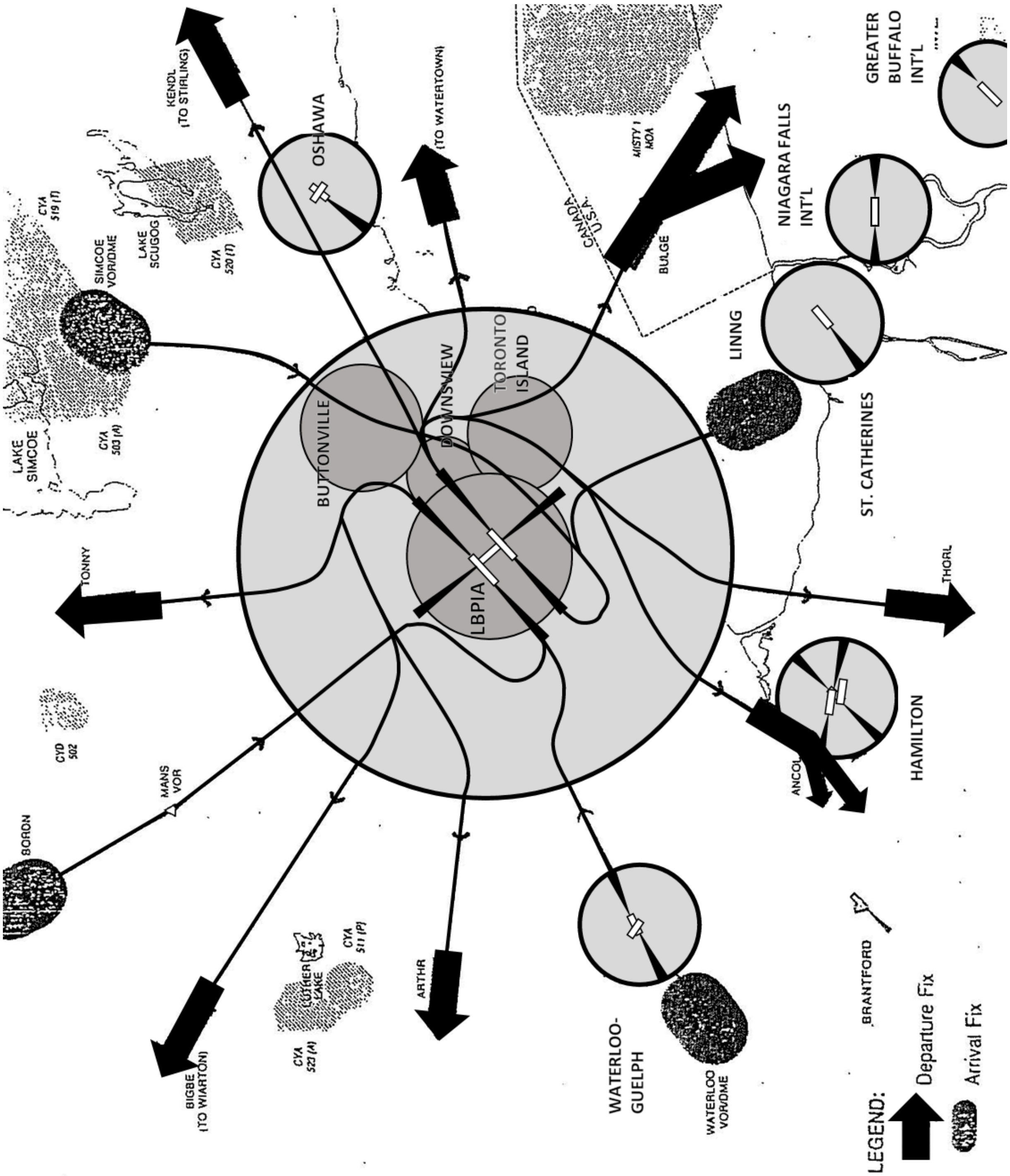


Figure 2.2 *Airspace Organization in the LBPIA Terminal Area Showing Arrival and Departure Paths for Simultaneous Operation of 06L and 06R*

The principal components of landing and take-off operations are illustrated in Figure 2.3. In the upper diagram, “A” identifies the point at which the en-route controller hands-off an arriving to the tower arrival controller. An aircraft flies a route “B” specified by the arrival controller and crosses the outer marker “C” and flies the final approach “D” touching down at “E” (for a perfect approach), the so-called aiming point of the instrument landing system. Aircraft decelerate and exit when an appropriate exit speed has been achieved at a particular exit “G”. “F” represents the runway occupancy time of an aircraft.

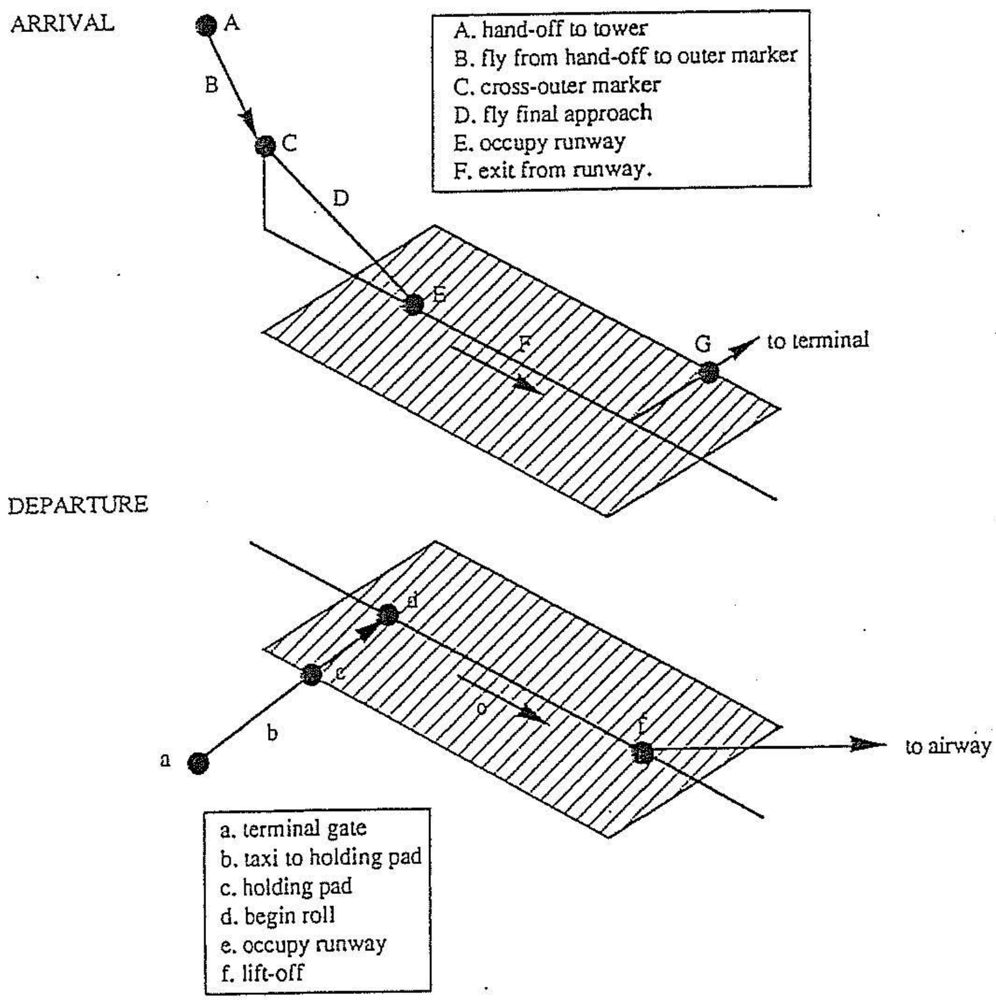


Figure 2.3 Components of Landing and Take-Off Operations

The lower part of Figure 2.3 shows the components of aircraft departure operations. Aircraft tare cleared to taxi from the terminal gate along a taxiway “b” to holding pad “c”. When aircraft are cleared for take-off they move to “d” and begin their roll occupying the runway “e” until lift-off occurs at point “f”. Departure procedures are normally standardized in that aircraft must fly on a predetermined route to a specific elevation where they join an en-route airway, where in some cases this departure route may be so arranged as to follow noise abatement requirements.

The arrival air traffic controllers build a queue of approaching aircraft along the approach path to the runway being used. This queue is built by merging aircraft into the queue from the approaching airways. The minimum separations between aircraft specified in a subsequent section must be satisfied along this approach path. If the demand rate is high, then the queue size increases and aircraft may have to fly relatively long approach distances at low speeds and high fuel consumption rates.

The number of runways, the airspace organization and the rules governing the minimum separation between aircraft determine the arrival and departure capacities of an airport. The capacity analysis of runway systems is described later in this chapter.

2.4 Design of Airside Facilities

Figure 2.4 shows the typical airside components that make up an airport facility. The major design decisions that have to be taken in connection with the planning of airside facilities are:

(i)

(ii)

(iii)

(iv)

(v)

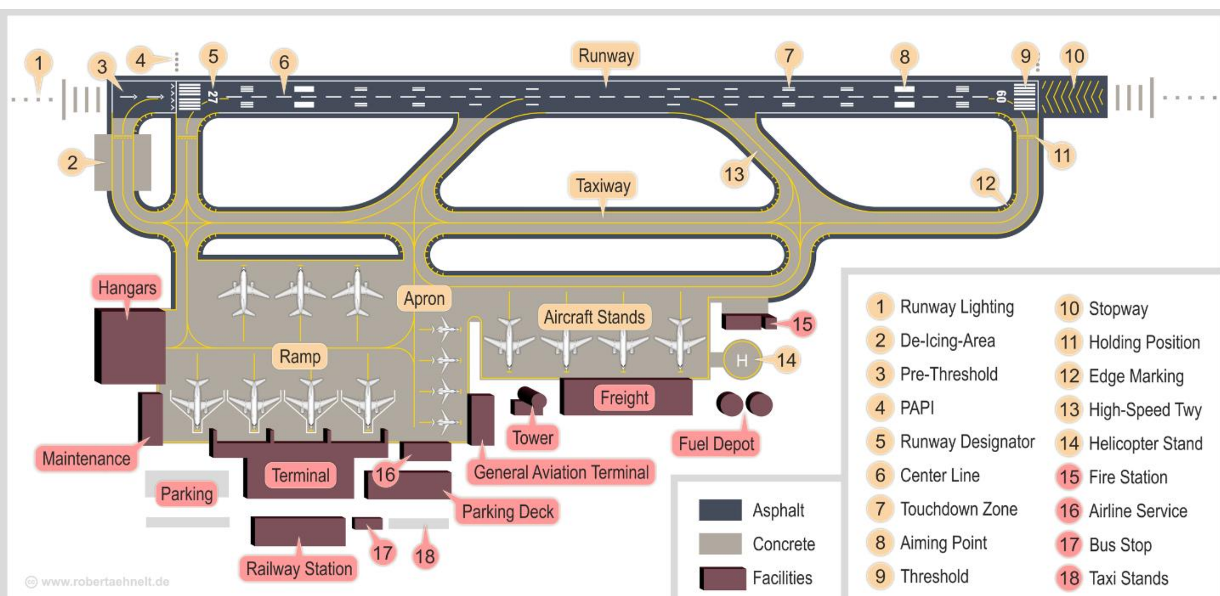


Figure 2.4 Airside components

2.4.1. Runway Labelling

Figure 2.5 illustrates the runway arrangement at LBPIA where there are five runways: 05-23, 06L-24R, 06R-24L, 15R-33L, and 15L-33R [GTAA 16]. Runways are labelled such that there is a two-digit number located at each end of the runway. Runway numbers are established by rounding the compass bearing of the runway centreline to the nearest ten degrees and then dividing by ten as viewed by the direction of approach. If the result is a single digit, then the number is preceded by a zero. If an airport has multiple parallel runways, the runways are assigned suffixes L, C, and R to designate left, centre, and right runways, respectively. The complete airside arrangement at LBPIA is presented in Figure 2.6.

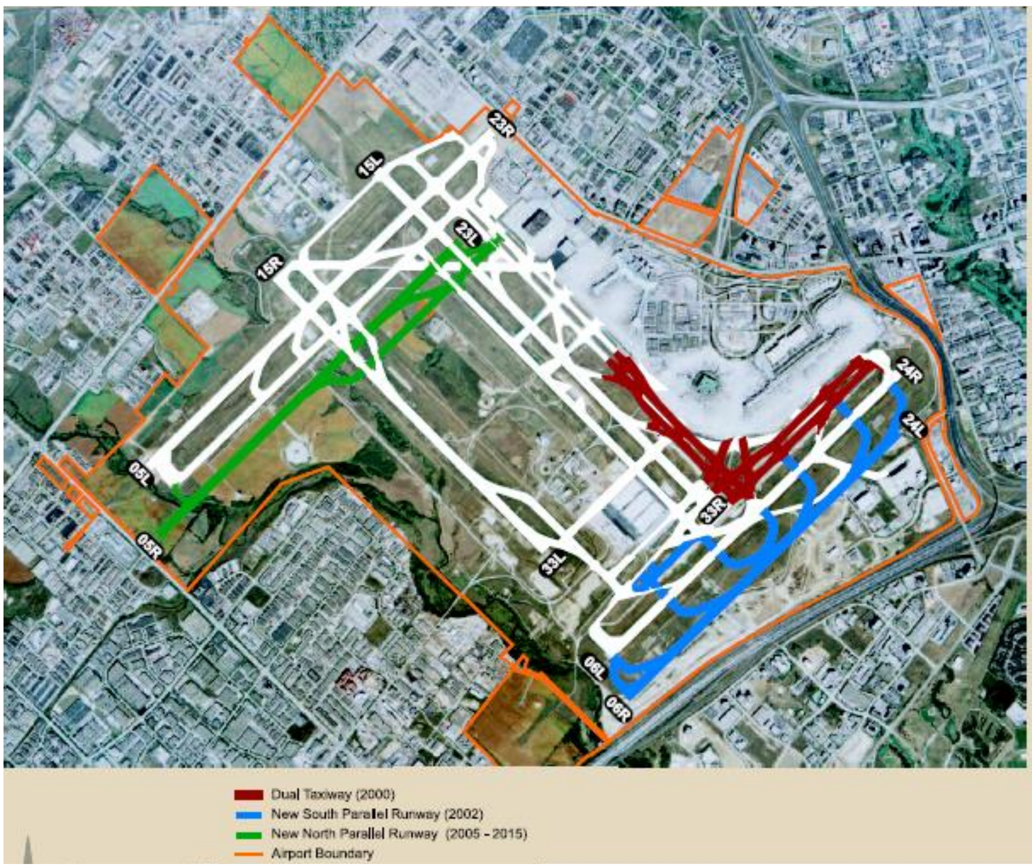


Figure 2.5 Runway Arrangement at LBPIA

Figure 2.6 Airside Arrangement at LBPIA (LBPIA Master Plan 2008-2030)

Example: Runway Naming

Based on Figure 2.7, what are the correct names for the runways marked **"a"** and **"b"**?

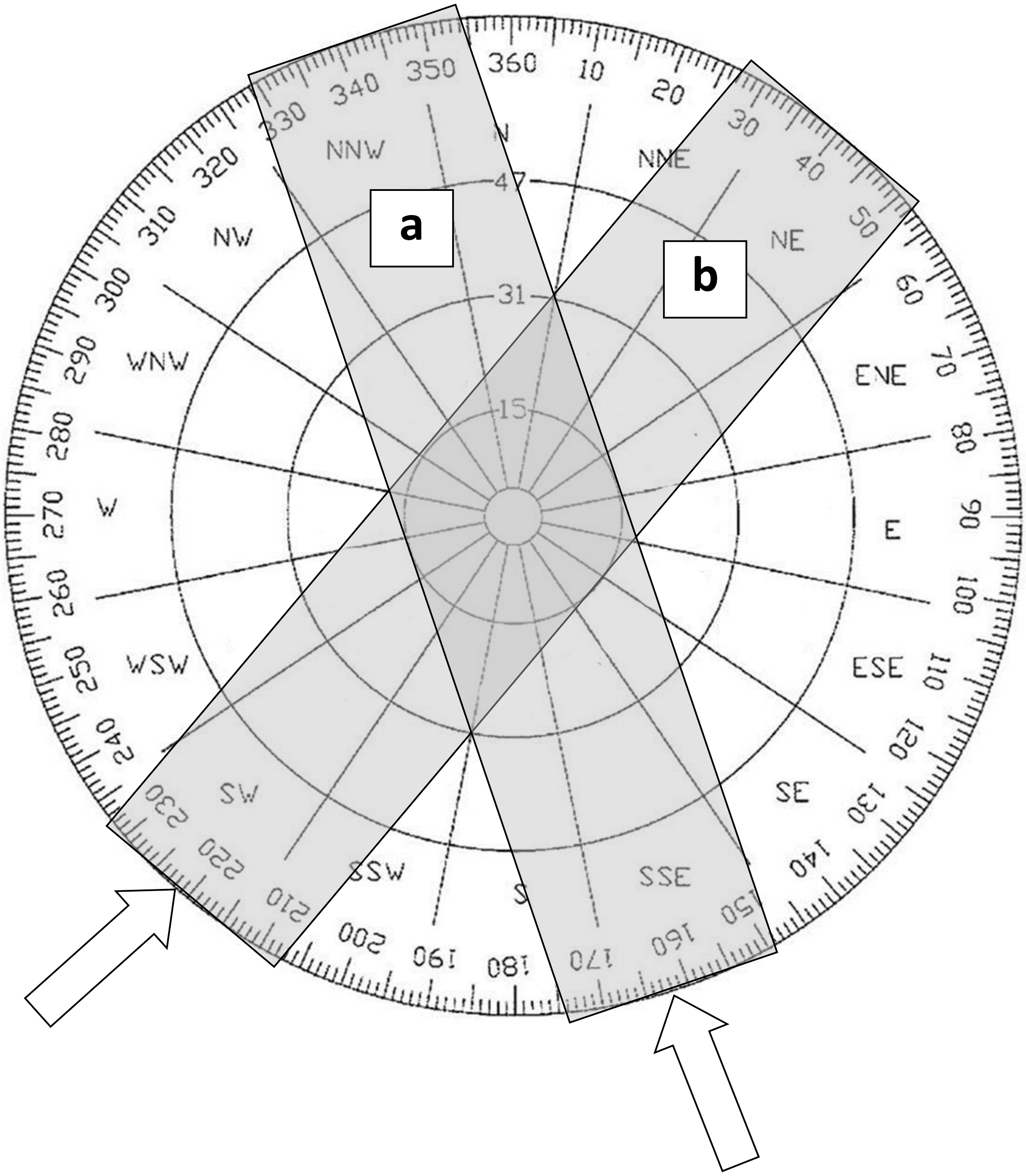


Figure 2.7 Runway naming example

Furthermore, airports located close to the north pole have a suffix T added to their runway number to indicate that the runway number is based on the true bearing, as opposed to the magnetic bearing [TC 93]. The true bearing is used in those cases since the magnetic north is not stationary and having a magnetic reading becomes increasingly difficult to maintain as you approach either of the magnetic poles [BBC 02].

2.4.2. Orientation of Runways

Decisions about the orientation of runways are based on knowledge of the prevailing winds at a particular site. The number of runways that should be provided in the main and cross-wind directions depends on the expected peak period aircraft operation demands. Runway length decisions are based on the controlling type of aircraft expected to use an airport and the length of trips to be flown by these aircraft. Long trip lengths require large fuel loads and greater runway lengths must be provided in order to allow aircraft to accelerate to higher speeds in order to develop the higher lift needed for take-off. The locations of taxiway-exits along runways influence the time that landing aircraft occupy a runway and this, in turn, influences the runway capacity. Finally, the relationship of terminals to runway ends influences taxiing patterns and the fuel consumption of arriving and departing aircraft.

2.4.3. Wind Analysis

Commercial aircraft operations are normally subjected to a maximum lateral wind speed component of 15 nm/h and a tailwind constraint of 5 nm/h (*1nm/h = 1 knot ≈ 1.15 mi/h*). A typical standard for commercial airports is that runways should be oriented so that aircraft may land at least 95 percent of the time with cross wind components not exceeding 15 nm/h when visibility criteria are satisfied. It is important to note that when the speed of the wind is given in mi/h, it is usually a simplification of the real unit (nm/h or knots).

Figure 2.8 summarizes the wind direction data for a particular location and indicates that the most common wind direction is from the SSE, followed by the NNW and SE. The contours of velocity for 5, 15, 31 and 47 nm/h are shown on the diagram, and the percent of the winds of a given range of velocities blowing from a given direction are shown in the segments of the diagram. That is, winds with velocities between 15 and 31 nm/h from the south blow for 2.2 percent of the time.

The structure of a wind rose diagram may be illustrated further by considering the wind direction and speed represented by the bold line vector illustrated in Figure 2.9. The speed components of this vector along the runway centreline and normal to the centreline may be calculated as illustrated in the diagram. The headwind component for aircraft landing on runway 27 would be about 20 nm/h and the crosswind component would be about 35 nm/h. This cross-wind component would be too high for landings on runway 27 and on runway 09. If the crosswind component were less than or equal to 15 nm/hr then it would enable a landing to occur. However, in this case, the tailwind component would need to be verified. If the tailwind is greater than 5 nm/hr, then the landing and takeoff would be reversed as there are no restrictions on headwinds.

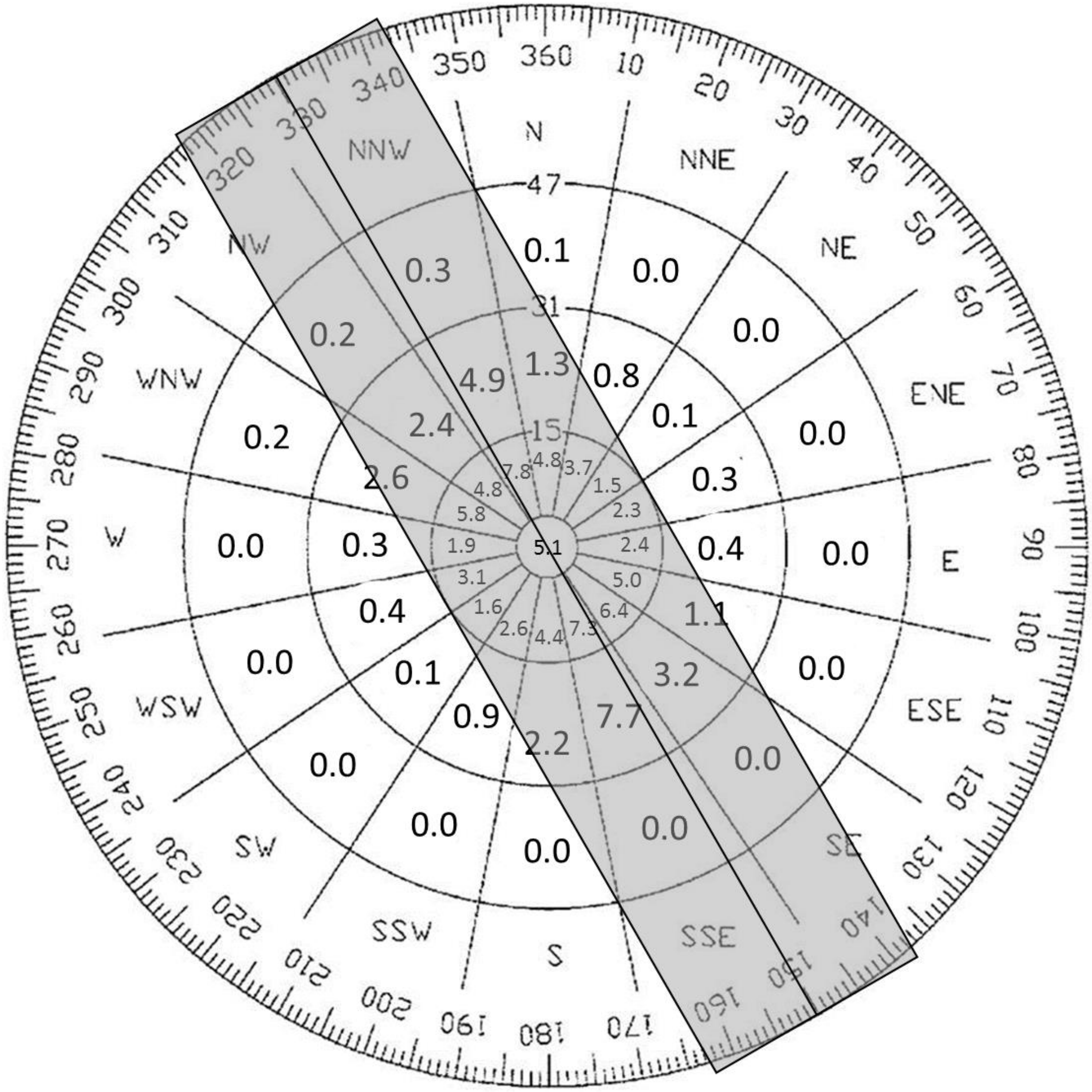


Figure 2.8 Wind rose Diagram

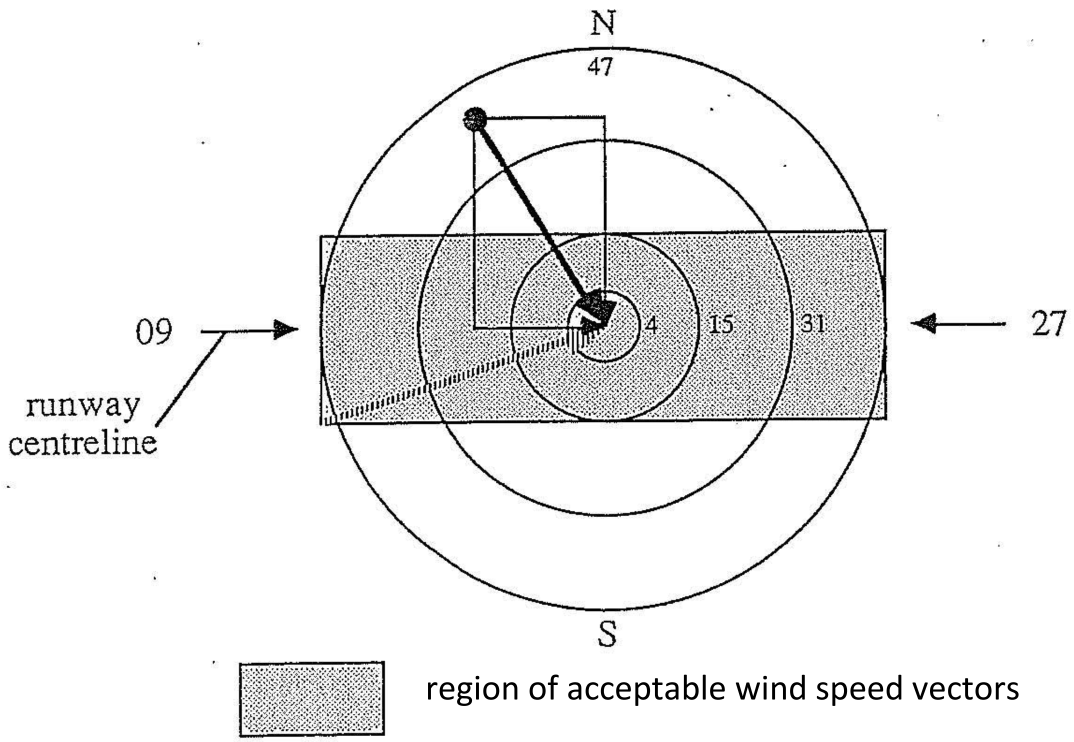


Figure 2.9 Representation of Wind Speed and Direction Vectors

The shaded area on Figure 2.9 is the region that defines the set of acceptable wind speeds and directions for aircraft operations on runway 09-27. This region has a width of 30 nm/h (15 nm/hr either side of the runway centreline). The hatched wind vector represents an extreme wind speed and direction within the acceptability region with a velocity of about 50 nm/h from the ESE. The lateral wind speed component is equal to 15 nm/hr and aircraft would have to land on runway 27 because the tailwind speed component would be greater than acceptable for landings on runway 09. Any wind vector within the shaded area would have an acceptable crosswind component and an acceptable tailwind component from at least one runway end.

The percentage of wind speeds and directions covered by this region indicates the percentage of the time that runway 09-27 could be used for aircraft operations. If a wind rose segment (defined by a pair of compass point lines and a pair of wind speed contours) is partially covered by an acceptability region, then the percentage contained in the segment is pro-rated according to the portion of the area covered by the acceptability region.

The optimum runway orientation may be established as illustrated in Figure 2.8. Imagine a transparent strip with its centre line coincident with the runway centre line, and the distance between the outside lines representing twice the allowable cross wind component (ie.30 nm/hr). This transparent strip may be rotated about the centre of the wind rose until the sum of the percentages bounded by the strip is a maximum. Winds up to 15 nm/hr are acceptable to any runway orientation, and this accounts for the 30 nm/h width of the strip.

Example: Runway Orientation

Given the orientation shown in Figure 2.8, what percentage of the time would you expect to be able use runway 15-33?

It should be recognized that the runway orientation may be shifted a few degrees in either direction since the bulk of the percentages are within the 30 nm/hr speed contour and common, therefore, to any runway orientation.

Airport planners are also interested in determining the capabilities of a particular runway orientation during restricted visibility conditions. A wind rose may be plotted for the particular restrictive conditions, and the proportion of the time that the runway can accept aircraft established.

A wind rose analysis may also be used to establish the proportion of the time throughout the year that two runways may be used simultaneously with the crosswind component not exceeding 15 nm/hr nor the tailwind component exceeding 5 nm/hr. Figure 2.10 illustrates this type of analysis for one operating configuration in which aircraft are landing from the west on runway 09 and from the north on runway 18.

The shaded area on Figure 2.10 shows the region of acceptable wind speed vectors, which would allow operations to occur safely on both runways. This hatched area represents the only sub-area common to the feasible regions for both runways, where these feasible regions are shown by the dashed lines. These regions are defined by the lateral wind speed constraint of 15 nm/h and the tailwind constraint of 5 nm/h.

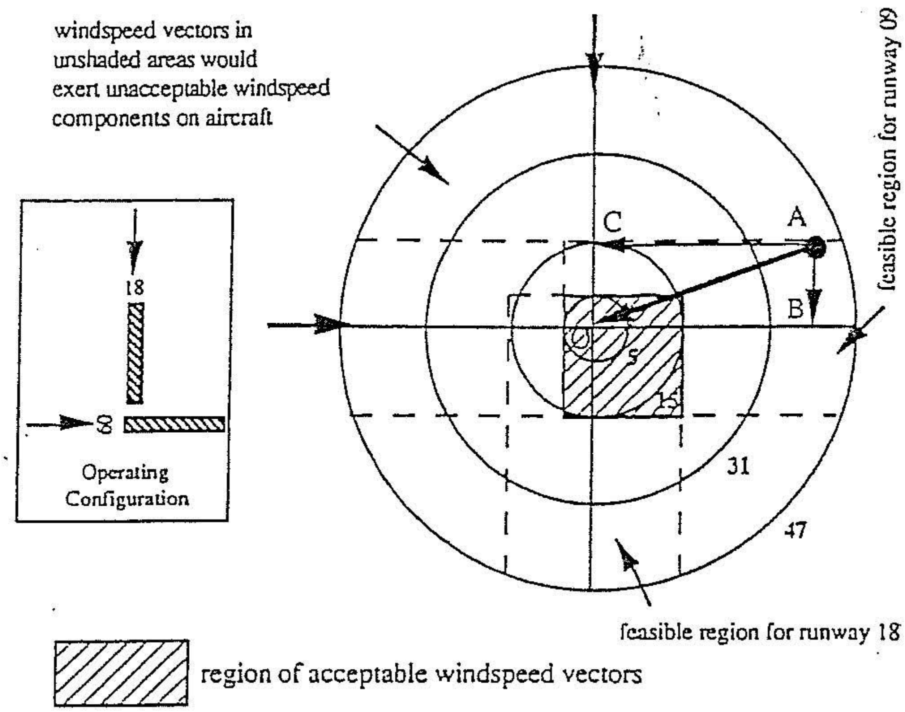


Figure 2.10 Wind rose Analysis of Joint Runway Operations

The unshaded area on Figure 2.10 shows the region of unacceptable wind speed vectors. For example, consider a wind represented by point A which would result in components AC and AB. The component AC would be an unacceptable lateral wind speed component for the aircraft landing on runway 18 but would be an acceptable headwind component for aircraft landing on runway 09. The component AB would not violate the lateral wind speed constraint for aircraft landing on runway 09 but would violate the tailwind constraint of aircraft landing on runway 18. That is, the hatched region shows the region of acceptable wind speed vectors for both aircraft movements.

2.4.4. Number of Runways

The previous analysis of wind speed and direction constraints has suggested that major airports require at least one main runway and one cross-wind runway in order to handle the wind conditions at any airport site. This section of the chapter examines the capacity characteristics of runways which are determined by the aircraft separation rules, the speed of aircraft and the runway occupancy times of landing and departing aircraft.

2.4.5. Runway Capacity

Runway capacity refers to the capability of a runway system to handle aircraft landings and departures. It is usually expressed in terms of the number of operations per hour for short-run capacity management, or the number of operations per year for long-run planning.

Saturation flow rate is usually defined as the maximum number of aircraft that can be handled in an hour under ideal conditions with a continuous stream of aircraft demanding service. When arrival and departure demands approach saturation capacity aircraft may experience significant delays. Runway capacity is usually defined in terms of practical capacity as illustrated in Figure 2.11, where a tolerable delay to aircraft is defined in order to establish the practical capacity. A U.S. Federal Aviation Administration (FAA) standard suggests that a runway has reached practical capacity when delays to arriving aircraft average four minutes during the normal two peak adjacent hours of the week.

2.4.5.1. First in First Out (FIFO) Queueing

One of the most useful probability density functions used in the applications of queuing theory to transport problems is the Poisson distribution. The variable in this distribution is the number of units which arrive in a particular time interval given a particular mean arrival rate.

The probability density function specifies the probabilities associated with each of 0 units arriving in (say) 5 minutes; 1 unit arriving in 5 minutes; 2 units arriving in 5 minutes, etc. Clearly, as the mean arrival rate increases (say from an average of 6 units per 5 minutes to 8 units per 5 minutes) the probabilities will increase for the larger number of arriving units.

The equation for the Poisson distribution is:

Equation 2.1

where:

In reviewing the structure of Equation 2.1 it is important to note that there is only one parameter (λ) required to specify a numerical form of the distribution and this is the mean of the distribution. Figure 2.11 illustrates the nature of the Poisson distribution for three different mean arrival rates (5, 10, & 30 ops/h), where these are for aircraft arrivals demanding to land on a single runway per one-hour time period. The sequence of diagrams illustrates how the probability density function shifts to the larger numbers.

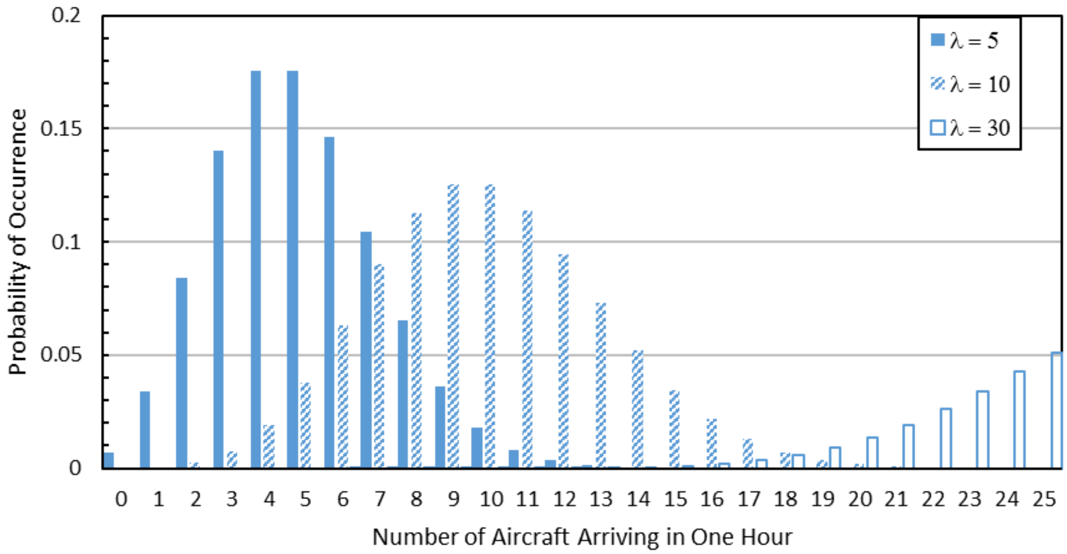


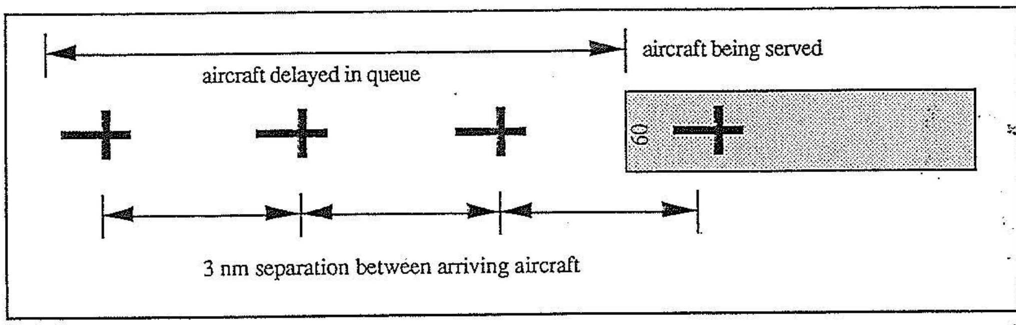
Figure 2.11 Poisson Distribution for Three Mean Arrival Rates

2.4.5.2. The Queuing Concept

Queuing theory deals with the behaviour at servers when the arrivals demanding service cannot all be served at once. The concept is illustrated in Figure 2.12 (a) where four aircraft have arrived at a runway together and only one can be served at a time; the other three have to be stored in a queue until the runway is vacant. The diagram illustrates the so-called FIFO (first-in-first-out) queue discipline. The dynamics of queuing systems depend on the distribution pattern of arrivals and of service times. The assumption made in this very brief treatment of queuing systems is that arrivals are Poisson distributed and the service times are distributed in a negative exponential way. It is also important to recognize that Figure 2.12 (a) illustrates a single-server queue and not a uni-queue/multiple server system that is common in banks and other service industries, etc.

Figure 2.12 (b) shows graphically how queuing occurs when arrival rates are greater than processing rates. The vertical and horizontal components of the queue portion of the plot represent the instantaneous queue length and the total duration of queuing, respectively. It should be noted that unlike the queuing example shown in this section, Figure 2.12 (b) shows linear arrival and service rates.

(a)



(b)

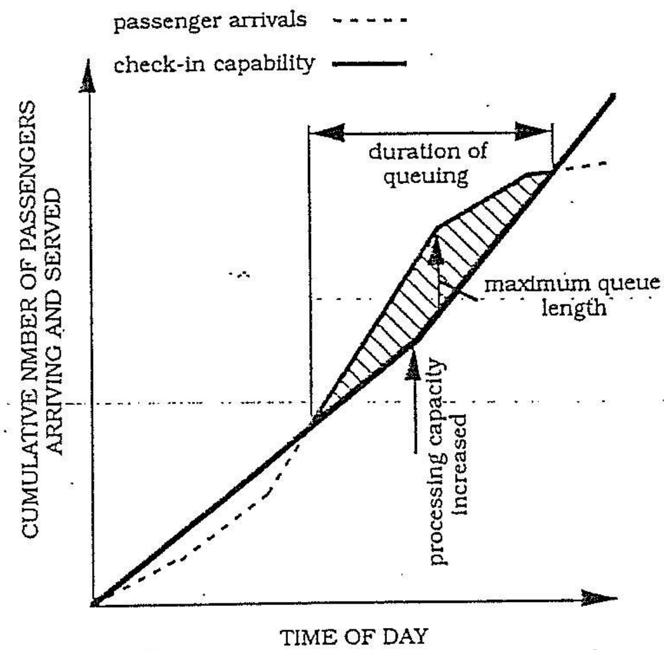


Figure 2.12 (a) Concept of a Simple Queue, (b) Graphical Representation of Queuing

Since the Poisson distribution has only one parameter it is usual to define the arrival rate parameter as λ and the service rate parameter as μ.Table 2.3 shows the theoretical results for a single server FIFO system with Poisson arrivals and exponential service times for steady state conditions. Many of the conditions are specified in terms of p = λ/μ which is called the traffic intensity factor which must be less than 1.0, otherwise the queue or waiting line would eventually become very large if λ were sustained over a long time period.

Table 2.3 Single Server Queuing Formulae for Poisson Arrivals and Exponential Service Times

	Queuing Model	Description of Model
1	$p(n) = (\rho)^n (1 - \rho)$	p(n) = probability of having exactly n vehicles in system
2	$\bar{n} = \dfrac{\rho}{1 - \rho}$	$\bar{n}$ = average number of vehicles in system
3	$\bar{q} = \dfrac{\rho^2}{1 - \rho}$	$\bar{q}$ = average length of queue
4	$\bar{d} = \dfrac{1}{\mu - \lambda}$	$\bar{d}$ = average time spent in system
5	$\bar{w} = \bar{d} - \dfrac{1}{\mu}$	$\bar{w}$ = average waiting time spent in queue

Example: Runway Capacity

Consider the case for a runway which has a service capability of handling 50 aircraft arrivals per hour; i.e. μ = 50 ops/h. Assume that a mean arrival rate of 40 ops/h exists during the three-hour afternoon peak arrival period. The formulae in Table 2.3 may be used to calculate some of the characteristics of this system.

1. probability of having exactly four aircraft in the system (i.e. one on the runway plus three in the approach queue)

2. average number of aircraft in the system

Example: Runway Capacity (continued)

3. average queue length

4. aircraft average time in the system

5. average time in the queue

Aircraft operations are subject to a number of Air Traffic Control (ATC) rules in order to ensure safety and these rules are summarized in Table 2.4. Arrival separations are presented in terms of nautical miles. The minimum separation is also shown in Table 2.4. The term heavy aircraft applies to wide-bodied aircraft such as the A 380 and the B-777. Inspection of the table entries shows that when the leading aircraft is a heavy aircraft then the separation is increased to 5 nm unless the trailing aircraft is also a heavy aircraft. This increased separation is to ensure that any wing-tip air turbulence created by the leading aircraft has time to dissipate. Inspection of these separation times shows that these are also increased when the leading departure is a heavy aircraft.

Operations on runways that are being used for arrival and departure operations are also subjected to an arrival-departure separation of 2 nm. This is also shown in Figure 2.14.

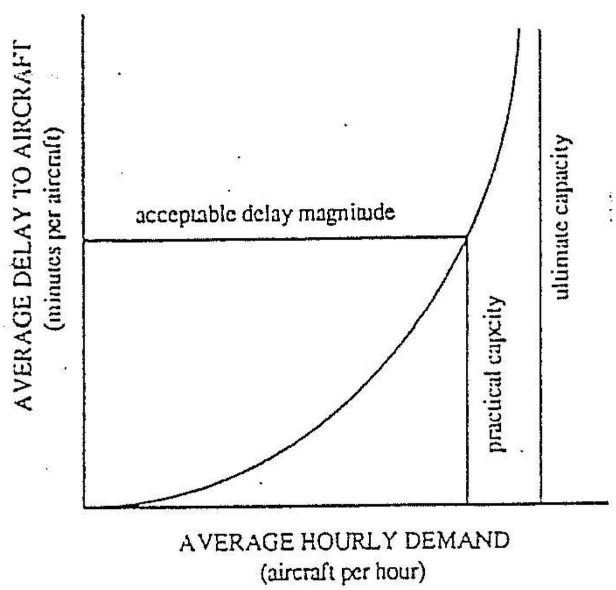


Figure 2.13 Average Delay to Arriving Aircraft Vs Arrival Rate

Table 2.4 Air Traffic Control Separation Rules: Minimum Separation of Arriving Aircraft (nm)

Trailing Aircraft	Leading Aircraft		
	Small	Large	Heavy
Small			
Large			
Heavy			

Table 2.5 shows the average arrival and lift-off speeds for the various aircraft categories along with typical runway occupancy times. The runway occupancy times are the times required for an aircraft to decelerate to an acceptable speed and exit from a runway. Runway occupancy times have an important influence on the departure capacities that may be achieved on runways being used for arrivals and departures, an effect which is illustrated later in this section.

Table 2.5 Aircraft Speeds and Runway Occupancy Times

Aircraft Type	Speed (nm/h or kts)		Runway Occupancy Time (secs)	
	Landing	Lift-Off	Landing	Take-Off
Small				
Large				
Heavy				

The aircraft distance separation rules are illustrated in Figure 2.14 for take-off separation, landing separation, and arrival-departure separation. As well, the runway cannot be occupied by more than one aircraft at any time.

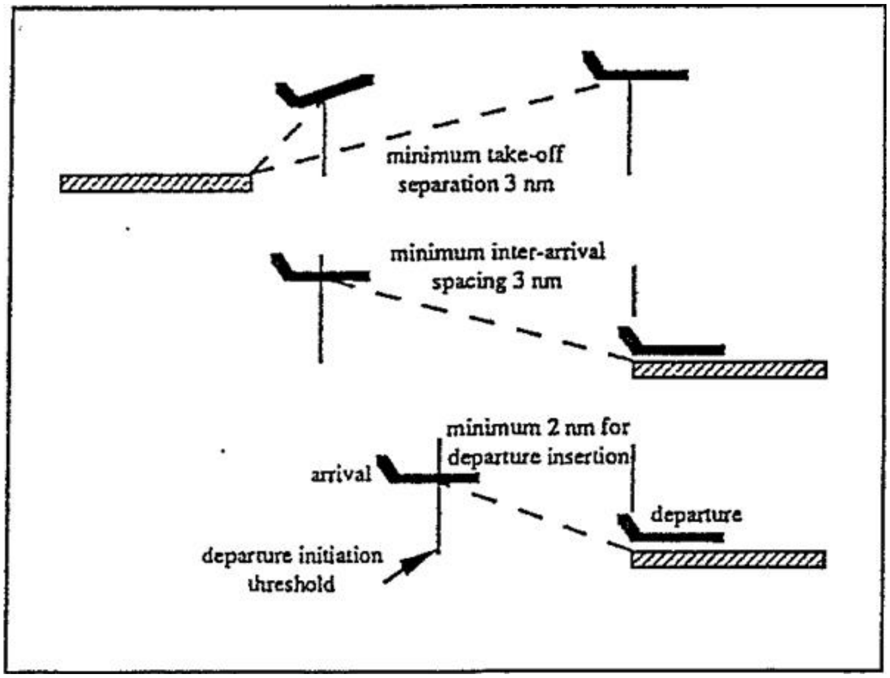


Figure 2.14 Aircraft Distance Separation Rules

The saturation flow rates of airport runways (the ultimate capacity of Figure 2.13) may be established using space-time diagrams and Figure 2.15 shows a space-time diagram for a typical situation in which an aircraft is landing at a speed of 124 nm/hr and which occupies the runway for 40 secs. The trajectory for a trailing aircraft is shown which arrives at the runway threshold 87secs later than the first aircraft. This means that if a departure were attempted, the second arriving aircraft would be closer than 2 nm. If the gap between the arriving aircraft was opened, then a departure could be successfully inserted between the two arriving aircraft. This would mean a time gap between arriving aircraft of 90 secs, or an arrival capacity of 3600/90 = 40 arrivals per hour.

There are a variety of other factors influencing saturation flow rates and these include (i) the characteristics of demand including arrival patterns and the mix of aircraft types (e.g., heavy commercial aircraft such as B747, DC 10, large aircraft such as B757 and general aviation aircraft) using an airport, (ii) the weather conditions, (iii) the layout and design of the runway and taxiway system, (iv) any limitations imposed by noise control considerations, and (v) the variability in aircraft, pilot and ATC.

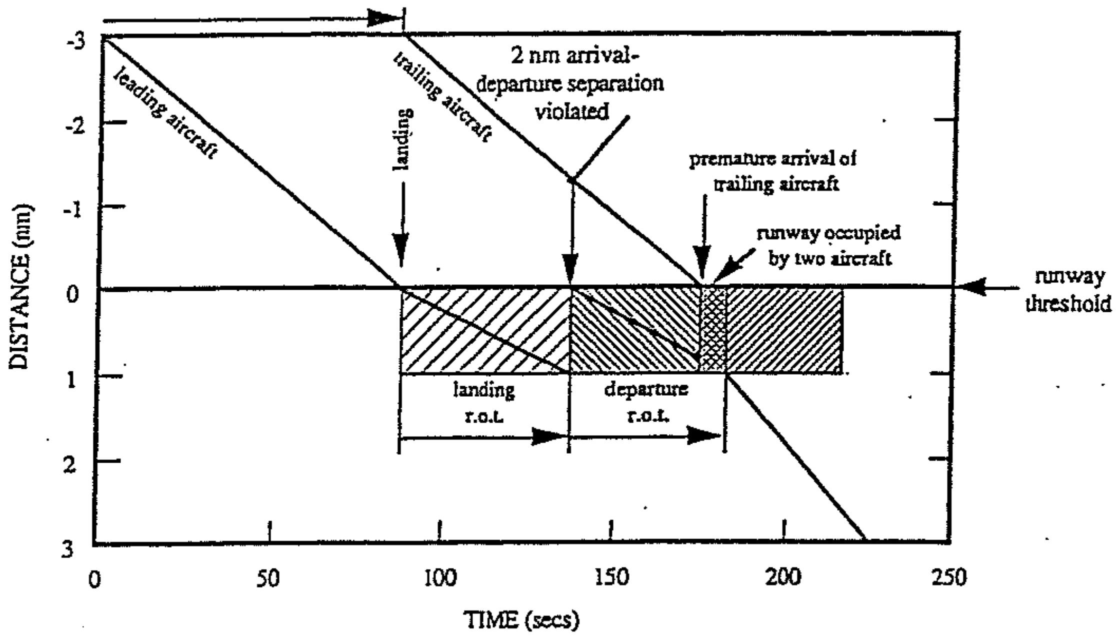


Figure 2.15 Space-Time Diagram for Runway Operations

Large airports have to handle a mix of aircraft types and the separation rules have to be adjusted. Different approach speeds and the potential impacts of wing tip air turbulence created by large wide-bodied jets on smaller aircraft are considerations. The length of the ILS (Instrument Landing System) common path (part D in Figure 2.3) is usually from 4 to 8 nm and ATC schedules arrivals at the gate such that the minimum aircraft separation standards will not be violated while aircraft are on the common path. The length of the common path affects runway capacity since larger separations will be required when a faster heavy aircraft follows a slower aircraft. Air traffic controllers increase distance separations between aircraft to ensure that unsafe overtaking does not occur, and the runway occupancy constraint of a single aircraft is not violated.

Example: Space-Time Diagram

A single runway is being used for simultaneous landings and departures by the following, repeating sequence of aircrafts: Large (L), Small (D), Heavy (L), Heavy (D). Each aircraft has an arrival runway occupancy time (rot) as shown in Table 2.5. How long does it take this sequence to complete? Construct a space/time diagram. (Consider the sequence to begin when the first plane is 3nm from landing and to be complete when the last plane is 3nm from the end of the runway)

Plane	Type	Arr/Dep	R.O.T. (s)	Arrival/Departure Speed (nm/hr)	Arrival/Departure Speed (nm/s)

Action	Time

Capacity Charts

The U.S. FAA conducted a major study of capacities at airports throughout the U.S.A. and has published capacity charts which reflect the effects of the following variables:

- Aircraft mix
- Runways serving arrivals and departures
- Touch-and-go operations
- Differences in exit taxiway configurations (location and turning radius),
- Environmental conditions (VFR, IFR)
- Runway configurations and conditions

Aircraft mix is defined in terms of four aircraft classes and these are:

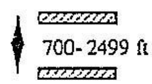
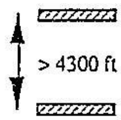
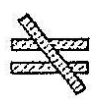
- Class A: small engine aircraft (<12,500 lbs),
- Class B: small twin-engine aircraft (<12,500 lbs),
- Class C: large aircraft (12,500 lbs to 300,000 lbs),
- Class D: heavy aircraft (> 300,000 lbs).

The capacity charts are defined in terms of an aircraft "mix index" which is determined by the percentages of aircraft in classes C and D as follows:

Mix index = % aircraft in class C + 3 x % aircraft in class D

Table 2.6 summarizes a sample of the type of information published in this handbook for four runway configurations and five mix index magnitudes. Inspection of the capacities for the single runway show that the capacity varies from 59 ops/h under IFR for the lowest percentage of Class C and D aircraft to 50 ops/h for very high concentrations of class C and D aircraft. This reflects the increasing distance separation that is required between heavy aircraft which reduces capacity. The high capacity under VFL for heavy concentrations of lighter aircraft is due to their ability to fly at separations less than those require under IFL.

Table 2.6 Capacities for Various Conditions-Runway

Runway Configuration	Runway Configuration Diagram	Mix Index (%C+3D)	Hourly Capacity (ops/h)	
			VFR	IFR
Single Runway		0-20 21-50 51-80 81-120 121-180	98 74 63 55 51	59 57 56 53 50
Dual Lane Runways		0-20 21-50 51-80 81-120 121-180	197 145 121 105 94	59 57 56 59 60
Independent IFR Parallels		0-20 21-50 51-80 81-120 121-180	197 149 126 111 103	119 114 111 105 99
Parallels plus Crosswind Runway		0-20 21-50 51-80 81-120 121-180	197 145 126 111 103	62 63 65 70 75

Example: Space-Time Diagram (continued)

Using the space/time diagram developed previously, what would the hourly capacity of that runway be?

2.5 Types of Aircraft

There are several types of aircraft with various loading configurations available in the aviation industry for the transportation of both passengers and cargo. Aircraft can be characterized by their main gear type and wheel configurations. Typical gear configurations and wheel assemblies are shown in Figure 2.16 and Figure 2.17, respectively.

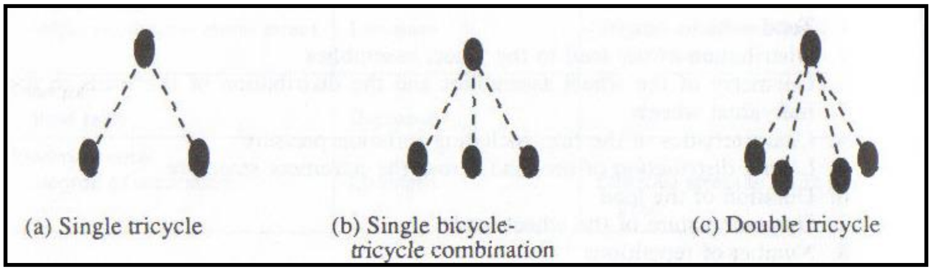


Figure 2.16 Typical gear assemblies [Haas 94]

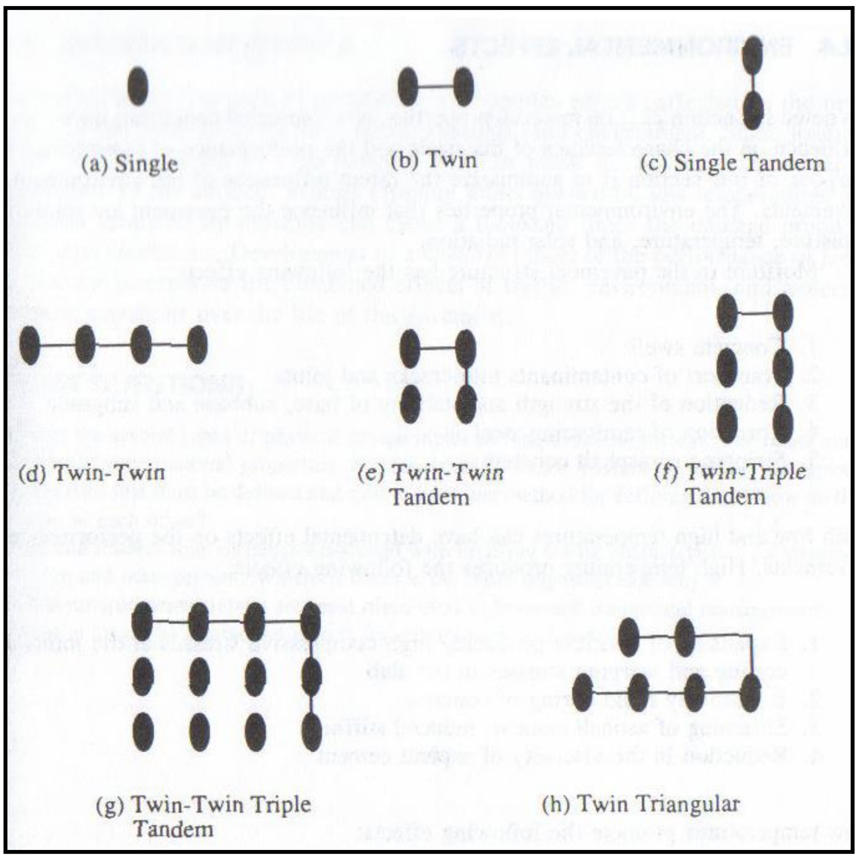


Figure 2.17 Typical aircraft wheel assemblies [Haas 94]

A single tricycle gear assembly consists of a nose gear and a set of main gears located on each wing. A single bicycle-tricycle combination gear assembly consists of a nose gear, a pair of wing gears and a belly gear along the centerline of the aircraft. The double tricycle gear assembly consists of a nose gear, a set of wing gears and a set of body gears or a nose gear and two sets of wing gears. Typically, it is assumed that the nose gear carries only 5% of the aircraft load, with the remaining 95% being equally distributed among the main gears [ACPA 01]. However, gross weights on the gears are a function of the contact area and tire pressure, which is subsequently discussed.

Figure 2.18 shows a landing Boeing 747. The plane has a double tricycle alignment. The nose gear is a twin assembly, while the two wing gears and two body gears are each a twin-twin tandem assembly. The Airbus A380 is a large plane which was first used in commercial service in 2007. The A380 has a similar double tricycle alignment, but the body gears are twin-triple tandems, to distribute the higher maximum mass of the plane (max take-off weights, 747: 447,700kg, A380: 590,000kg).

Figure 2.18 Boeing 747 landing gear alignment

The most common types of gear configurations are the single, twin (also known as dual), and the twin-twin tandem (also referred to as the dual tandem). The new large aircraft (NLA) typically have a twin-triple tandem gear configuration, which are also referred to as a triple dual tandem (TDT) or 3 duals in tandem or twin-tridem/dual tridem wheel configuration. The gear and wheel configurations affect how the aircraft load is distributed through the wheel contact area to the underlying pavement.

The aircraft's most critical load is its static load when it is parked on the apron while is it fully loaded with passengers, cargo and fuel. The aircraft load decreases as the aircraft makes its way from the apron at the terminal gate to the taxiway system and onto the runway primarily due to the accelerated speed before take-off. Therefore, the maximum taxi-off weight (MTOW) or the maximum gross weight (MGW) is commonly used as the design load for pavement design purposes. [TC 04a]

The tire contact area for an aircraft tire is calculated by dividing the single wheel load by the tire inflation pressure. Aircraft tire pressures are available from the aircraft manufacturers. The shape of the tire footprint area is generally assumed to be an ellipse. The major axis, which runs parallel to the direction of travel, is assumed to be 1.6 times the minor axis. Furthermore, the minor axis, which is perpendicular to the direction of travel, is calculated to be 0.894 times the square root of the contact area. [Boeing 04]

2.6 International Airport Classification

The U.S. FAA and the International Civil Aviation Organization (ICAO) categorize aircraft in terms of six different classes based on wingspan and tail height or wingspan and outer main gear wheel span as shown in Table 2.7. Examples for each of these groups/classes are provided in Table 2.8 [Boeing 07].

Another form of aircraft classification is the Aircraft Load Rating (ALR) used in Canada. The ALR is based on their gear load, tire pressure, and load distribution. A rating scale of one to thirteen is used, whereby an aircraft with an ALR of one would create minimal damage to a pavement structure; and an aircraft with an ALR of thirteen could cause extensive damage to a pavement structure. The characteristics of standard gear loads for single, dual and dual tandem wheels are shown in Table 2.9 shows the weights, field lengths and cruising speeds for contemporary aircraft.

Table 2.7 FAA and ICAO Airport Classifications

FAA Airport Design Airplane Design Group			ICAO Aerodrome Reference Code		
Group	Airplane Wingspan m (ft)	Tail Height m (ft)	Code	Aeroplane Wingspan m (ft)	Outer Main Gear Wheel Span m (ft)
I	< 15 (49)	<6 (20)	A	< 15 (49)	<4.5 (15)
II	15(49) – 24 (79)	6 (20) – 9 (30)	B	15 (49) - <24 (79)	4.6 (15)- < 6 (20)
III	24 (79) – 36	9 (30) – 14 (45)	C	24 (79)- < 36 (118)	6 (20)- <9 (30)
IV	36 (118) – 52	14 (45) – 18 (60)	D	36 (118)- <52 (171)	9 (30)- <14 (46)
V	52 (171) – 65	18 (60) – 20(66)	E	52 (171)- <65 (213)	9 (46)- < 14 (46)
VI	65 (214)– 80 (262)	20 (66) – 24(80)	F	65 (213)- <80 (263)	14 (53)- <16 (53)

2.7 Runway Length Requirements

The runway length requirements for take-offs are a function of the wing loading of an aircraft, the wing area, the lift coefficient (approximately proportional to the angle of attack) and the air density (a function of air temperature and elevation ASL). Take-off occurs when the lift exerted on the wings is greater than the wing loading created by the aircraft, fuel and pay load weights, and lift is given by:

Equation 2.2

Table 2.8 Boeing Commercial Aircraft – Design Group/Codes (FAA/ICAO)

	FAA Airplane Design Group						ICAO Aeroplane Design Code					
Aircraft	I	II	III	IV	V	VI	A	B	C	D	E	F
707 (all)				X						X		
717-200			X						X			
720				X						X		
727 (all)			X						X			
737-100			X						X			
737-200			X						X			
737-300, -300W			X						X			
373-400			X						X			
737-500			X						X			
737-600			X						X			
737-700, -700W			X						X			
737-800, -800W			X						X			
737-900ER, -900ERW			X						X			
BBJ			X						X			
BBJ2			X						X			
BBJ3			X						X			
747SP					X						X	
747-100, 200, 300					X						X	
747-400, 400ER					X						X	
747-817-8F˚						X						X
747-LCF						X					X	
757-200, -200W				X						X		
757-300				X						X		
767-200				X						X		
767-300, -300W				X						X		
767-400ER				X						X		
777-200, 200ER, 200LR					X						X	
777-300, 300ER					X						X	
787-3˚				X						X		
787-8, -9˚					X						X	
BC-17/C-17A				X						X		
DC-8-43, 55				X						X		
DC-6-61, 71				X						X		
DC-8-52, 72				X						X		
DC-8-53, 73				X						X		
DC-9-15			X						X			
DC-9-21			X						X			
DC-9-32			X						X			
DC-9-41			X						X			
DC-9-51			X						X			
DC-10-10										X		
DC-10-30, 40										X		
MD-11										X		
MD-81, 82, 83, 88			X						X			
MD-87			X						X			
MD-90-30			X						X			

*747-8 and 787 data are preliminary

Innovations in structural design and new materials (as well as powerful jet engines) have allowed substantial increases in allowable wing loads over the past 70 years which has permitted much heavier aircraft to be designed and flown. The field length required for take-off is influenced by air density and higher air temperatures and higher airport elevations increase field length requirements. Landing field lengths are determined by landing speeds, acceptable deceleration rates for passenger comfort and the coefficient of tire-pavement friction that may be developed under various weather conditions.Table 2.10 summarizes the weights and FAR (Federal Aircraft Regulations) take-off and landing field lengths required by some of the current aircraft in commercial service.

Table 2.9 Standard Gear Loads [Argue 05, PWGSC 97, TC 02b]

Standard Gear Load (ALR)	Characteristics of Standard Gear Loads								Example of ALR Aircraft
	Single Wheel		Dual Wheel Gear			Dual Tandem Gear			
	Gear load (kN)	Tire Pressure (MPa)	Gear Load (kN)	Tire Pressure (MPa)	Tire Spacing (mm)	Gear Load (kN)	Tire Pressure (MPa)	Tire Spacing (mm)	
1	20	0.30							Beech 18
2	30	0.35							King Air
3	45	0.40							Lockheed
4	60	0.45	80	0.50	500				DC-3
5	80	0.50	110	0.60	550				Dash-7
6	110	0.55	130	0.65	600				Convair 440
7	140	0.60	170	0.70	650				DC-4
8			220	0.85	700				DC-9
9			290	1.05	750	440	1.10	650x1150	B-737
10			400	1.15	900	660	1.20	900x1500	B-767
11						900	1.55	1100x1650	DC-10
12						1120	1.80	1150x1650	L-1011
13						1380	1.80	1150x1650	B-52 Bomber

Table 2.10 Weights, Field Lengths and Cruising Speeds for Contemporary Aircraft

Aircraft Type	Weight (kg x 1000)		FAR Field Length (m)		Cruising Speed (km/h)
	Take-Off	Landing	Take-Off	Landing	
B747-400	364	261	3,400	2,150	950
DC10-30	252	183	3,200	1,810	950
Concorde	182	111	3,120	2,425	2,240
B767-200	136	123	1,715	1,430	965
B757-200	100	90	1,875	1,465	940
B737-500	53	52	2,015	1,600	920
DC9-50	55	50	2,400	1,420	885
BAe146-200	40	35	1,500	1,055	800

Figure 2.19 shows the variation in FAR runway take-off length for a B747 for changes in aircraft weight, airport elevation and air temperature. A number of criteria have been established for determining the length of a runway and these reflect the characteristics of the various aircraft as well safety considerations. During certification, an aircraft is required to demonstrate the field length requirements for (i) take-off to 11 m, (ii) completion of a take-off to 11 m with engine failure at a critical point in the take-off, (iii) stopping after aborting a take-off following an engine failure at the same critical point, and (iv) to stop after landing from a height of 15 m. Safety margins are then added which are typically 15% in the all-engine operating take-off case and 67% in the landing case. The ways in which field length are determined are summarized in Figure 2.20.

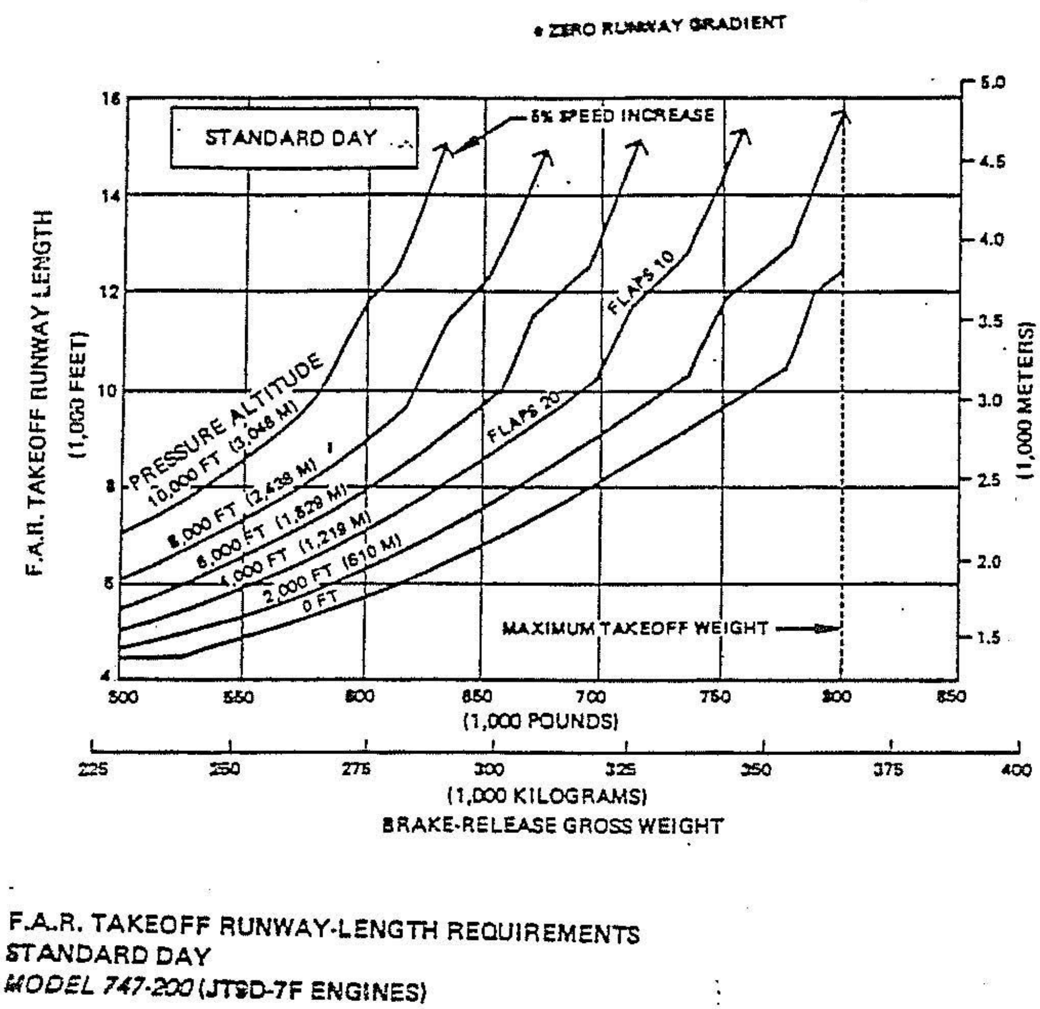


Figure 2.19 Runway Length Requirements for B747

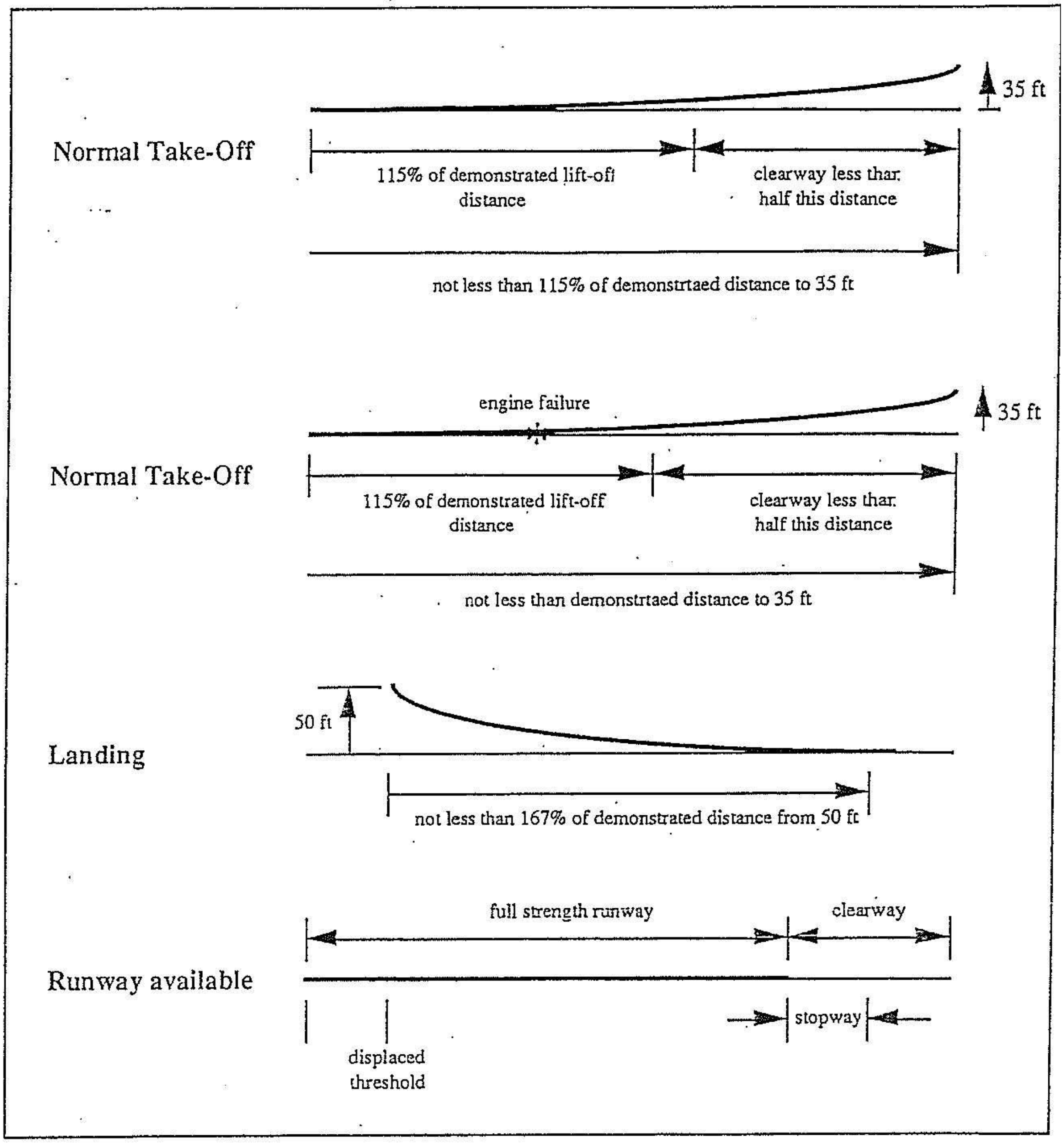


Figure 2.20 Field Length Determinants

2.8 Airport Configurations

Figure 2.21 illustrates some of the possible inter-relationships between runways, terminals and operations for various airport configurations. The diagrams illustrate arrangements geared to minimizing taxiing distances. For example, in Figure 2.21 the staggered runways may be used in either direction, but at any one time each runway is used exclusively either for take-offs or landings.

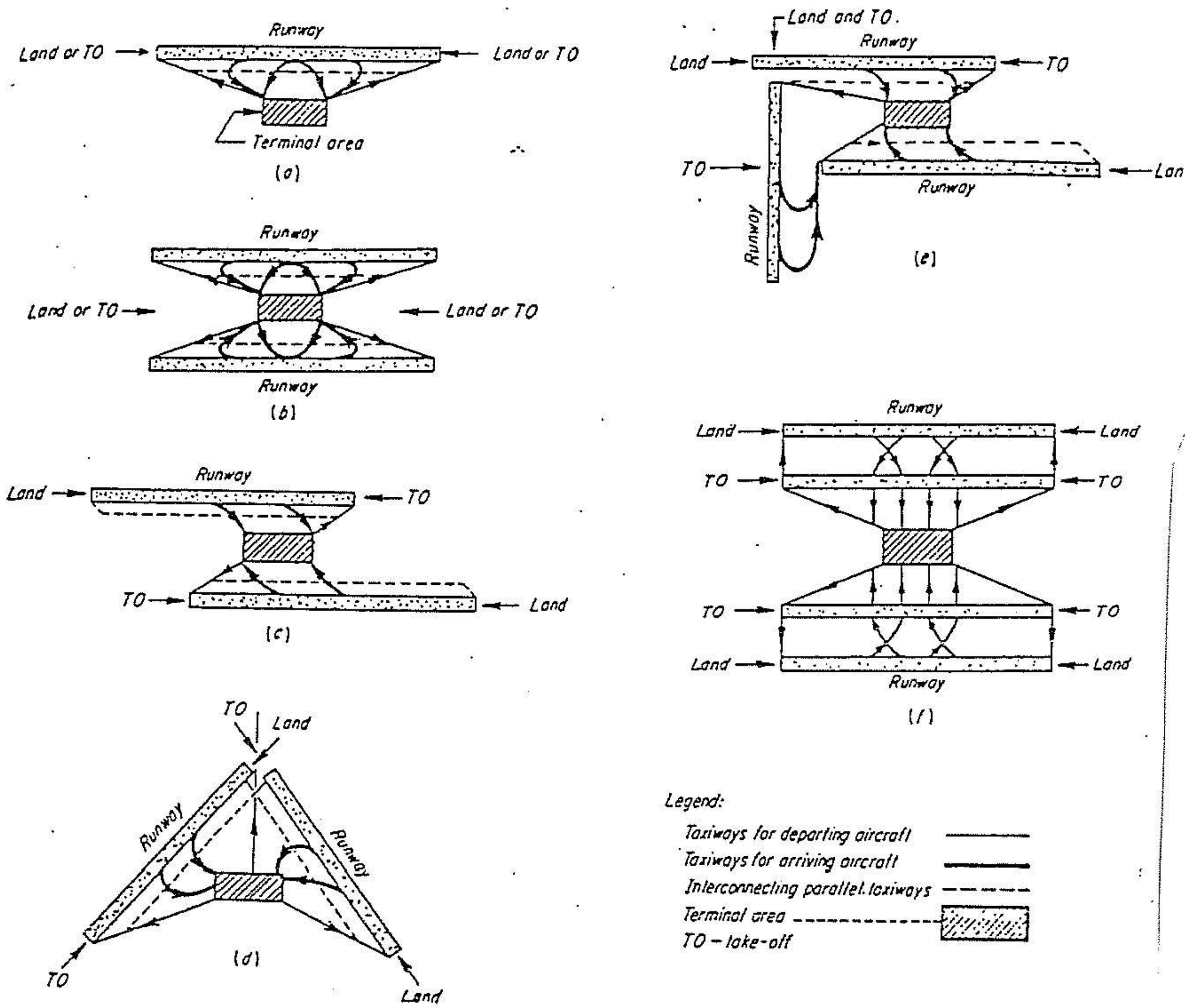


Figure 2.21 Typical Airport Configurations

2.9 Taxiway Design

According to Transport Canada practice, taxiways should be designated by a letter, letters, or a combination of letters followed by a number. Rapid exit taxiways, which are used to quickly remove aircraft from the runway and decrease runway occupancy time, should use an alpha/numeric system that should identify the taxiway to which they are connected. The numeric portion of the rapid exit taxiway should be odd numbers for exits serving easterly runways and even numbers for westerly runways. The intersection of a rapid exit taxiway and a runway should be between 25 and 45 degrees. [TC 93]

Examples of taxiway names include: Taxiway D (Delta) and Rapid Exit Taxiway W1 (Whiskey One). Taxiways connect the runways to the terminal area and service hangars. Exit taxiways have an important influence on runway capacity since their number, location and design influence the runway occupancy time of landing aircraft.

The southernmost runway at LBPIA is 06R-24L. This runway, which is 2743m long, has eight separate taxiways servicing it. Six of the taxiways are early exit taxiways by which the plane can exit the runway before reaching the end. Because of the naming convention of the airport, the taxiways are numbered Deltas (D). The numbering is summarized in Figure 2.22.

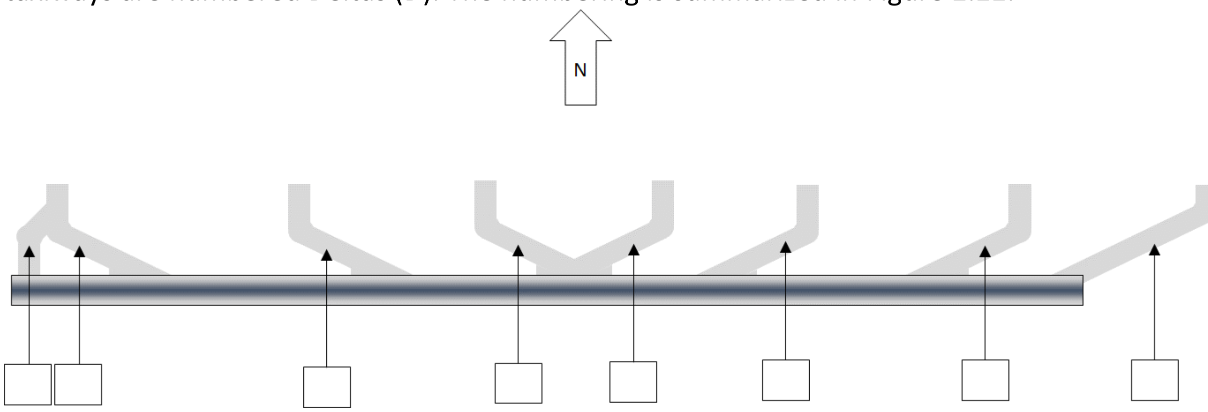


Figure 2.22 LBPIA's Runway 06R-24L

The space-time diagram shown in Figure 2.15 illustrates how landing runway occupancy time influences arrival and departure separation. Figure 2.23 illustrates the components of the aircraft landing process. The flare phase occurs between the runway threshold and the touchdown point. The lower diagram illustrates that the flare distance consists of the linear descending distance from the threshold to the touchdown aiming point plus the circular flare distance required for a smooth landing. The threshold crossing height is regulated at 15.24 m and the angle 2.5^o – 3^o is determined by the instrument landing system (ILS). This distance is typically about 450 m for turbojet aircraft.

The next largest component of the landing distance is the deceleration distance during which the aircraft decelerates from touchdown speed to exit speed. The touchdown speed is about 90% of the approach speed (200 km/h) and exit speeds are typically 100 km/h with a deceleration of about 0.15 g. This means that a deceleration distance of about 790 m would be required.

Figure 2.24 illustrates the relationship between exit geometry and exit speed and the left side of Figure 2.25 illustrates typical exit designs while the right side emphasizes the importance of understanding aircraft geometry. Calculations of this type provide a general idea of the most appropriate locations but field observations have shown a great deal of variability in touchdown distances and deceleration rates.

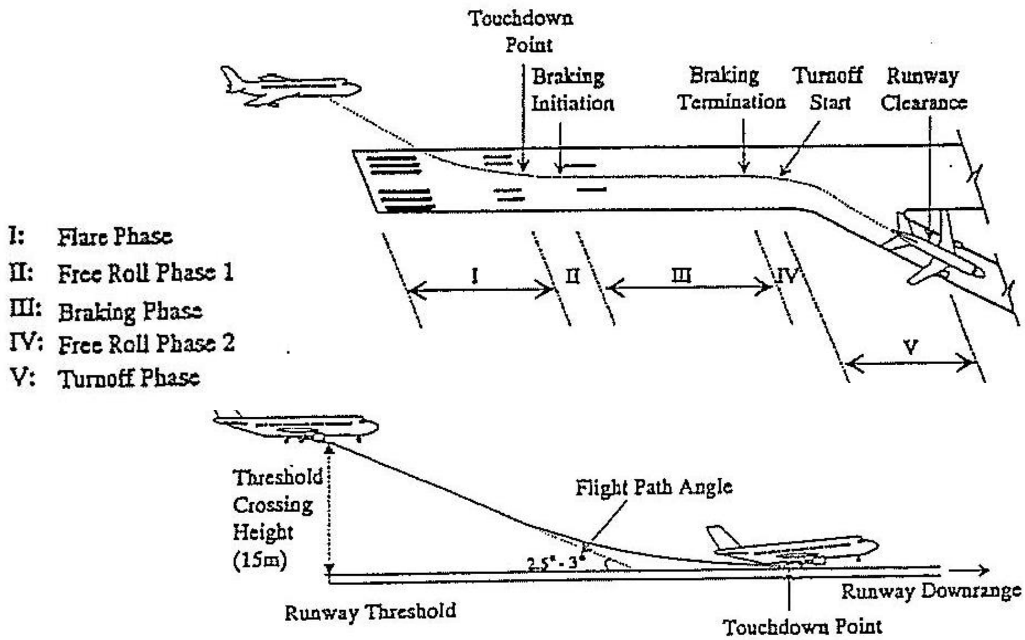


Figure 2.23 Components of Landing Process

It is normal practice to provide runway exits at about 750 m, 1200 m and 1900 m from the runway threshold. When the nose wheel of an aircraft follows a curved path, the centre of the undercarriage does not follow the trajectory of the nose wheel. In the design of taxiways and apron layouts the trajectory of the outer wheels of the aircraft undercarriage and the wing tips is of importance. In addition, the aircraft undercarriage deviates from the taxiway centre line. If the undercarriage wheels move too close to the edge of the pavement serious cracking may occur because of the extremely high loads imposed by very large aircraft. The longer the wheelbase of the controlling aircraft, the larger the fillet.

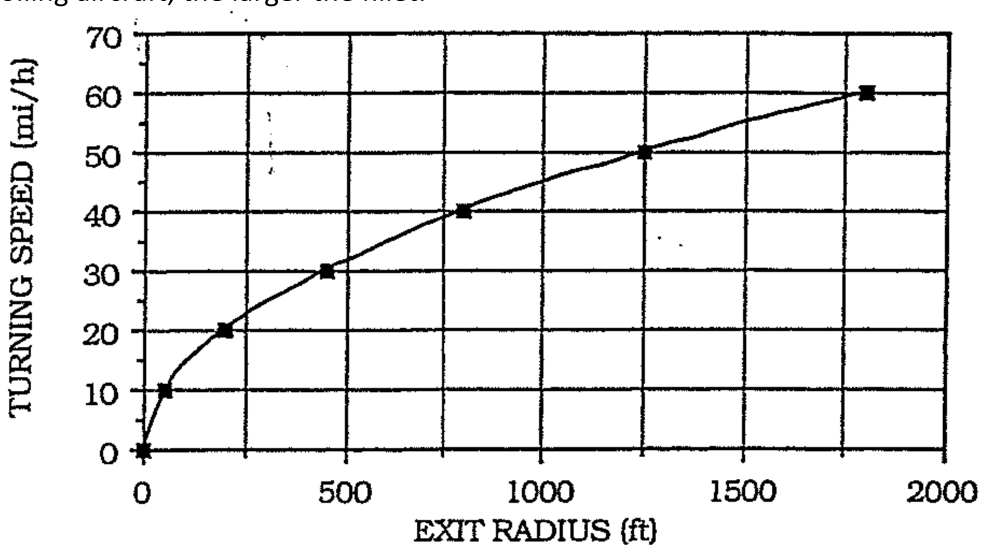


Figure 2.24 Allowable Turning Speed vs Exit Radius

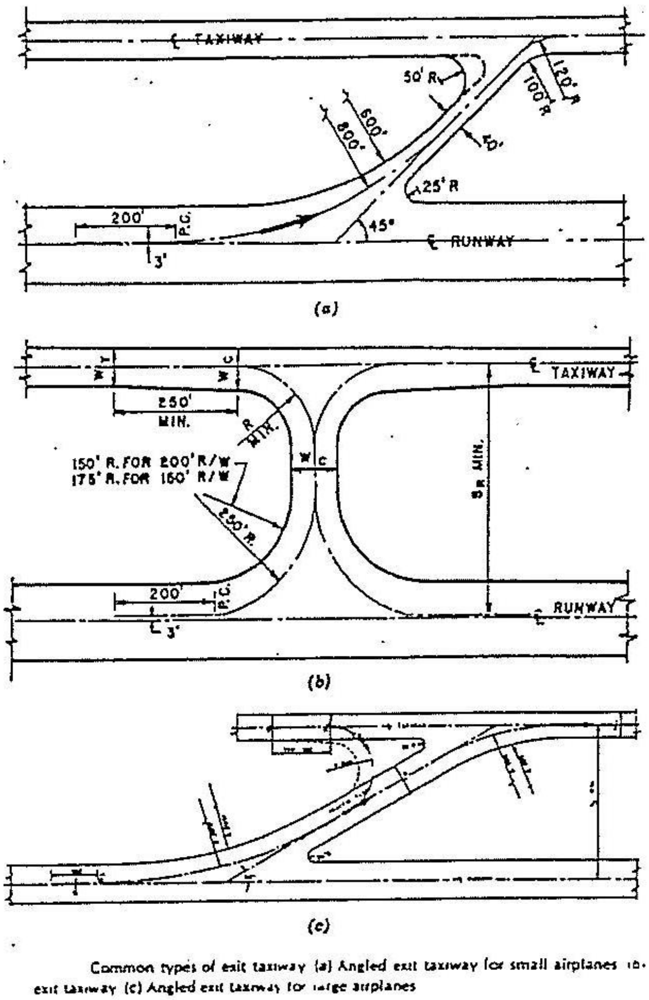
As larger aircrafts become more economical for various airlines and routes, the airside infrastructure must accommodate these changes. One instance is the radius for turns on taxiways. The inside radius of a curve must be such that a minimum clearance is maintained between the outside edge of the landing gear and the edge of the pavement. Some facilities were designed to accommodate planes which were classified as Group E, which have gear widths of 9-14m. Group F planes, with gear widths of 14-16m are becoming more common. [ICAO 05]

Figure 2.26 illustrates a plane which can easily traverse a curve on the left and one which cannot on the right. Several options can be used by the airport authority to address this issue:

1.
2.
3.

The choice largely depends on the space and cost restrictions at a given curve or airport.

A high-speed taxiway is designed with lighting or marking to define the path of aircraft, travelling at high speed (up to 60 kt), from the runway centre to a point on the centre of a taxiway. The high-speed taxiway is designed to expedite aircraft turning off the runway after landing, thus reducing runway occupancy time [TC 12].



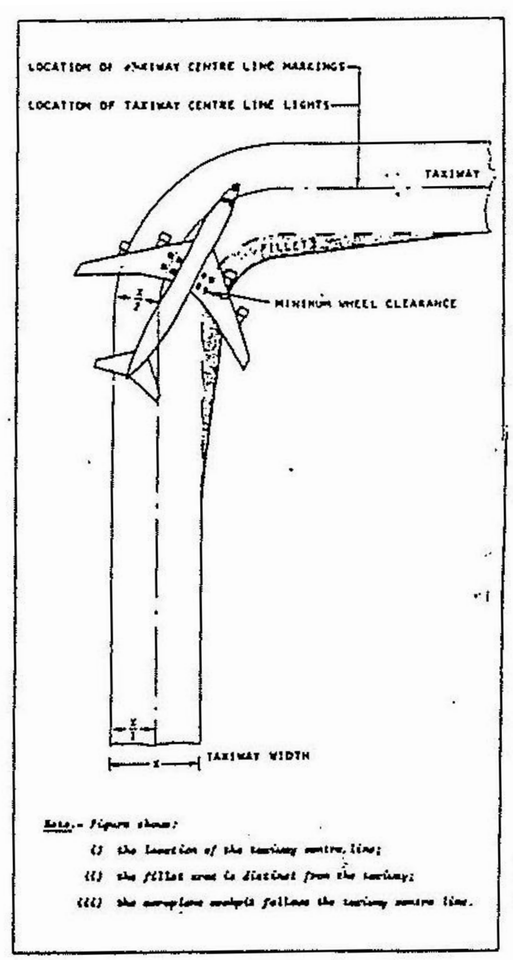


Figure 2.25 Common Types of Taxiway Exits and Taxiway Design Considerations (right)
Figure 2.26 Radius increasing for larger classification airplanes (left)

2.10 Environmental Impacts of Airports – Noise Impacts

Noise from airport operations is the most apparent adverse environmental impact generated by commercial airports. Other environmental impacts include air pollution from aircraft exhaust and the pollution of run-off from the air-side of airports which contaminated by hydrocarbons, rubber and the compounds used in de-icing fluids. The concern of this section is with the noise impacts of airports.

2.10.1. Basic Measures of Noise

The energy produced by any sound is transmitted through the air in waves and these are eventually received by the ear, where this transmission may be influenced by wind speed and direction, wind temperature, humidity and intervening objects. Ears are sensitive to a very wide range of sound pressure frequencies and the sound pressures over a band of frequencies is compressed into a decibel measure (dB) which is expressed relative to the quietest sound pressure that an individual with excellent hearing can perceive. That is, the decibel level of a sound pressure is given by:

Equation 2-1

where:

A sound pressure equal to this reference sound pressure has a dB magnitude of 0 dB while the loudest sounds that people can hear without pain have a dB magnitude of around 120 dB. The normal background noise in quiet sections of urban areas is about 50 dB. Normally the sound pressure level is determined using a sound meter.

The frequency of sound is also an important consideration since the ear has a differential sensitivity to sound pressures at different frequencies with the most sensitive range of the ear being between 1,000 to 2,000 Hz. Figure 2.27 illustrates the various weighting scales that have been used, where the A-weighting scheme is essentially flat between 1,000 Hz and 10,000 Hz.

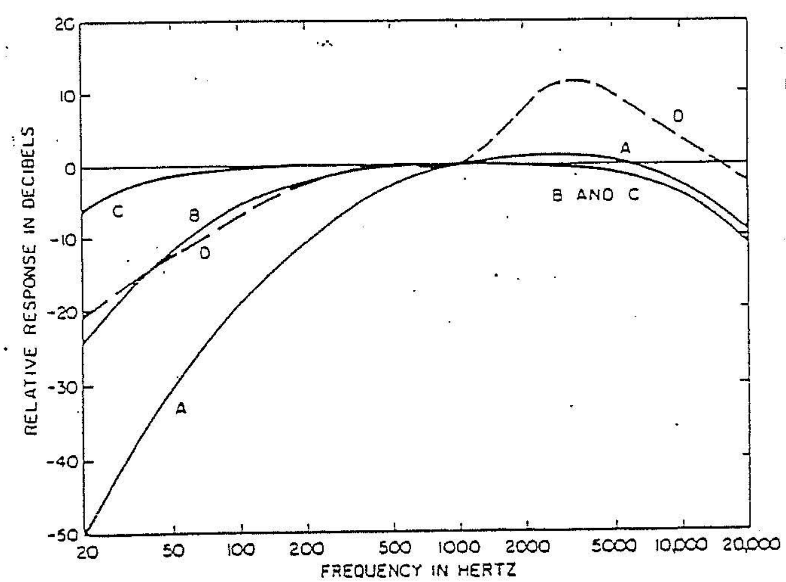


Figure 2.27 Sound Pressure Frequency Weighting Schemes

The audible frequency range is from about 20 Hz to a high of about 14,000 Hz. The basic dB scale has been modified to produce the so-called A-weighted decibel scale, dBA, and this scale filters the sound pressures and places greater weight on those that occur in the frequency ranges to which the ear is most sensitive and less weight to sound pressures at those frequencies to which the ear is not particularly sensitive such as those less than 500 Hz.

Figure 2.28 illustrates the dBA magnitudes of common sound sources and the diagram notes that the typical ambient noise level in urban areas is around 50-60 dBA.

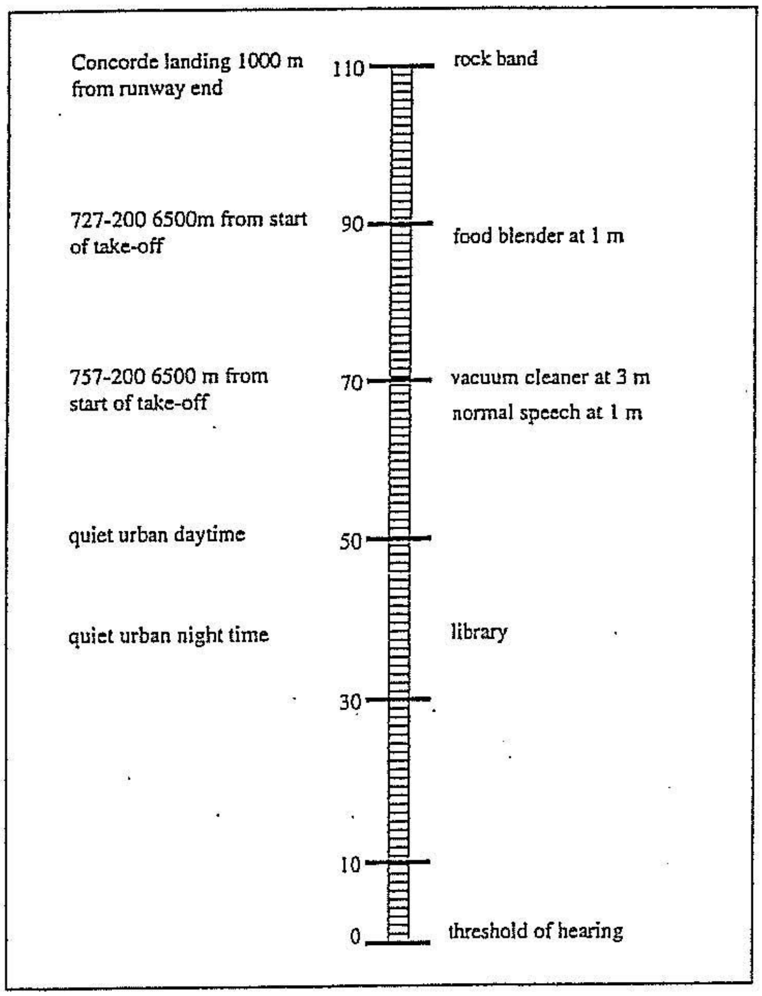


Figure 2.28 Typical A-Weighted Sound Pressure Levels

It should be recognized that because the decibel scale is a logarithmic scale, an increase of 10 dBA is perceived as a doubling of the sound pressure. A change of 5 dB in sound pressure is perceived to be significant while a change of about 3 dB is considered to be imperceptible. Because sound pressure levels (measured in dB's) are expressed on a logarithmic scale, they must be transformed to sound pressure levels, added and then re-converted to dB magnitudes as in Equation 2-2.

Equation 2-2

where:

For example, sound pressures received at a location X from three noise sources are 65dB, 72dB and 68dB. The resultant sound pressure would be:

2.10.2. Sound Exposure Level (SEL)

Estimating the annoyance to people of the noise generated by commercial aircraft operations is complicated by the fact that sound pressure generated at a single point by an aircraft increases as the aircraft approaches and then decreases as the aircraft moves away from the observer. The measure used to capture the noise impact of a single aircraft flyover is the Sound Exposure Level (SEL). It is the accumulation of the sound energy from the time that the A-weighted sound pressure is perceived to the time that the sound is no longer audible from the aircraft using the equal energy principle, and then normalized to a reference duration of 1 sec.

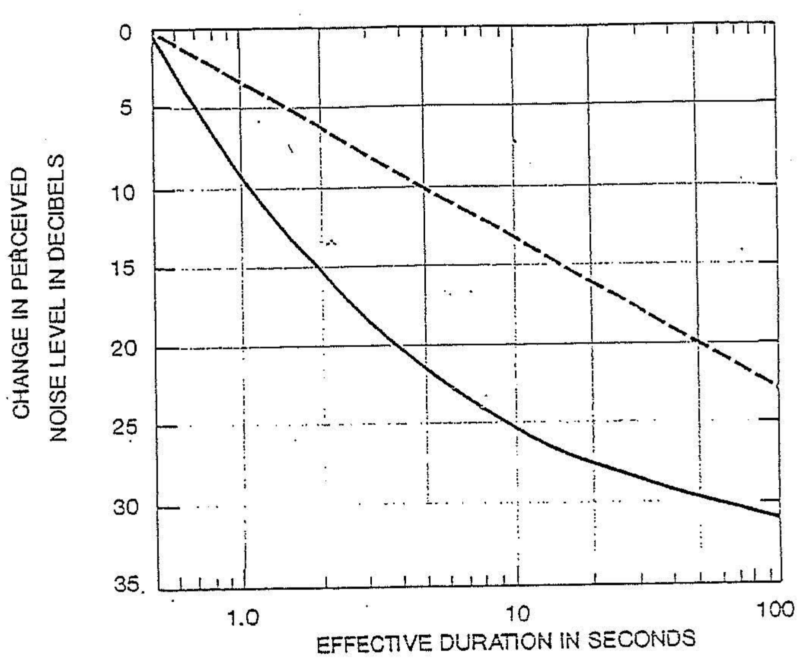


Figure 2.29 Relationship between Changes in Noise Level and Duration

The dashed line in Figure 2.29 illustrates the equivalent energy principle of sound exposure; two sounds are judged to be equally annoying if they produce the same noise dose over time. This principle requires that if the sound energy from a source is doubled then its duration must be halved if it is to be judged equally annoying. The graph shows, for example, that a change in perceived noise level from 10 dBA to 20 dBA (sound pressure increases by 10 times) then the effective duration increases from 5 secs to 50 secs (i.e. 10 times the duration); or vice versa.

Figure 2.30 illustrates this idea where the sound pressure is standardized to a one second dose using the equal energy principle which is illustrated in the previous numerical example. It should be noted that the A-weighted sound pressure of this one second dose is larger than the maximum dBA received by an observer and this is usually 7 to 12 dB's higher than the maximum dBA (Lmax) experienced by an individual.

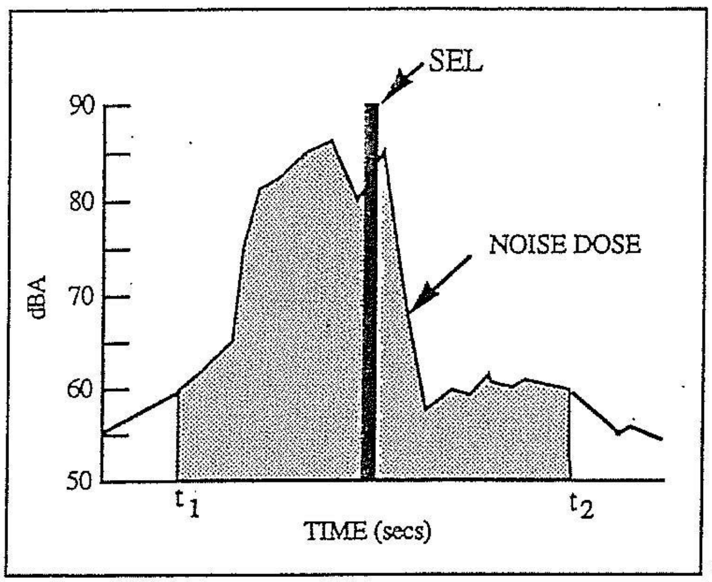


Figure 2.30 The Concept of Sound Exposure Level (SEL)

The SEL is calculated in a similar manner to calculating dB_x as shown in Equation 2-3. However, in this case the time associated with each interval must be taken into consideration.

Figure 2.31 illustrates a one-minute record of a flyover of a single aircraft. Calculate the SEL, given the following equation:

Equation 2-3

where:

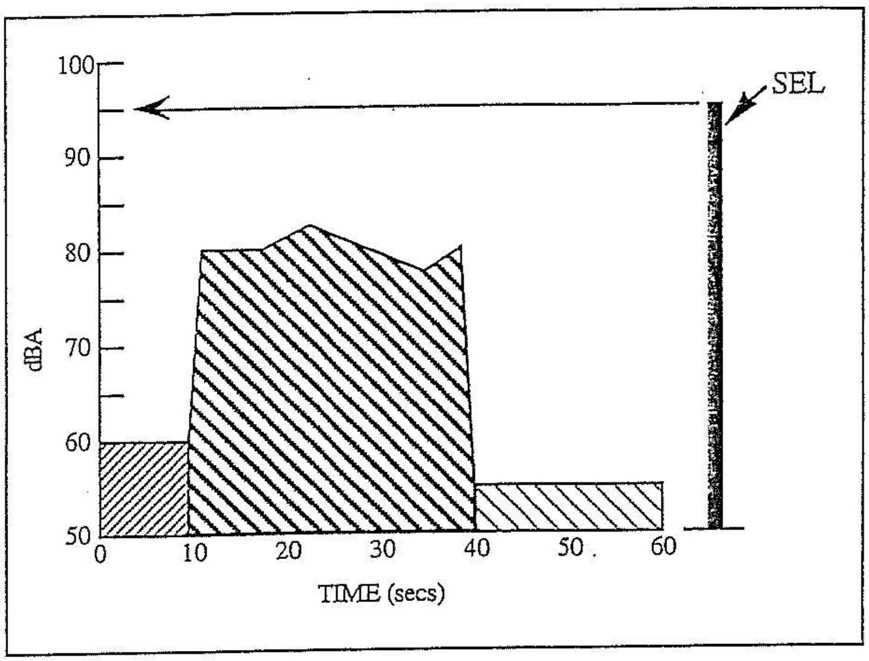


Figure 2.31 Conversion of Sound Energy to a One-Second Dose

SEL can also be calculated and provided as a summary value per plane.

Figure 2.32 illustrates the SEL magnitudes for a B747 at various distances from the flight track centreline and for various approach distances from the runway centreline for a 3° glide slope (which defines the elevation of the aircraft at various distances before the runway threshold). An individual function in Figure 2.32 (say for 140,000 ft from the runway threshold) shows the attenuation of noise with increasing distance (roughly an inverse square decay function); at about 15,000 ft normal to the flight track centreline the noise generated by the B747 aircraft has decreased to a SEL of 70 dBA.

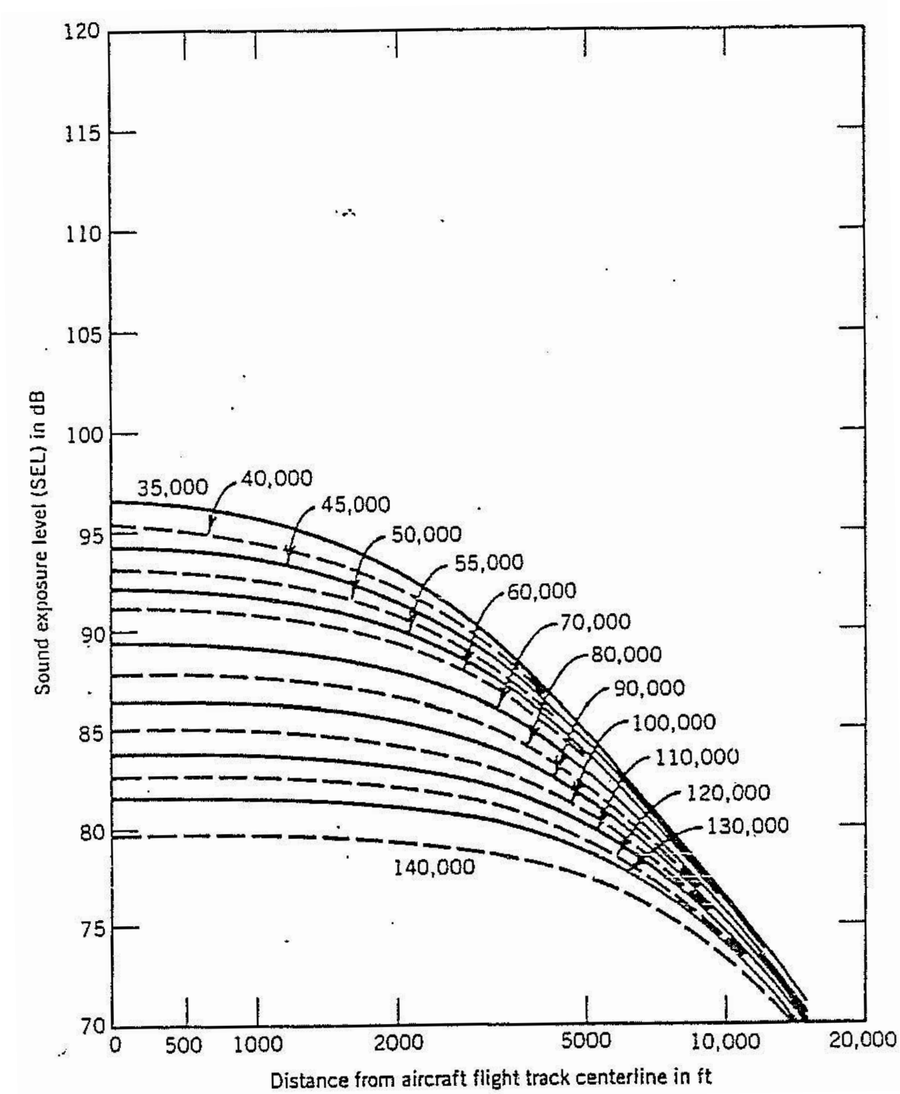


Figure 2.32 SEL versus Distance from Aircraft Track Centre Line for Various Distances from Runway Threshold.

2.10.3. Day-Night Equivalent Sound Level (L_{dn})

Airport operations normally occur over much of the day depending on noise curfew times imposed to protect surrounding residents from excessive noise during night time. Noise doses that occur between 2200 h and 0600 h are usually assigned a surcharge of 10 dBA to reflect the additional annoyance to surrounding residents of night time operations. The noise doses imposed by aircraft throughout a day are also summed by adding their sound pressures and the night time surcharges to obtain the day-night noise equivalent (L_{dn}) which is provided in terms of dBA. This approach is illustrated in Figure 2.33 for a hypothetical set of aircraft operations. The noise energy created by an individual flight is first converted to an SEL for a one second duration using information of the type illustrated in Figure 2.31. A sequence of SEL's for various aircraft type could then be converted to a sequence of one-hour SEL's and any noise premiums for night time operations added and these could then be converted to the L_{dn} for an entire day of operations in the same way that the sound energy in an individual flyover is converted to an equivalent one second noise dose.

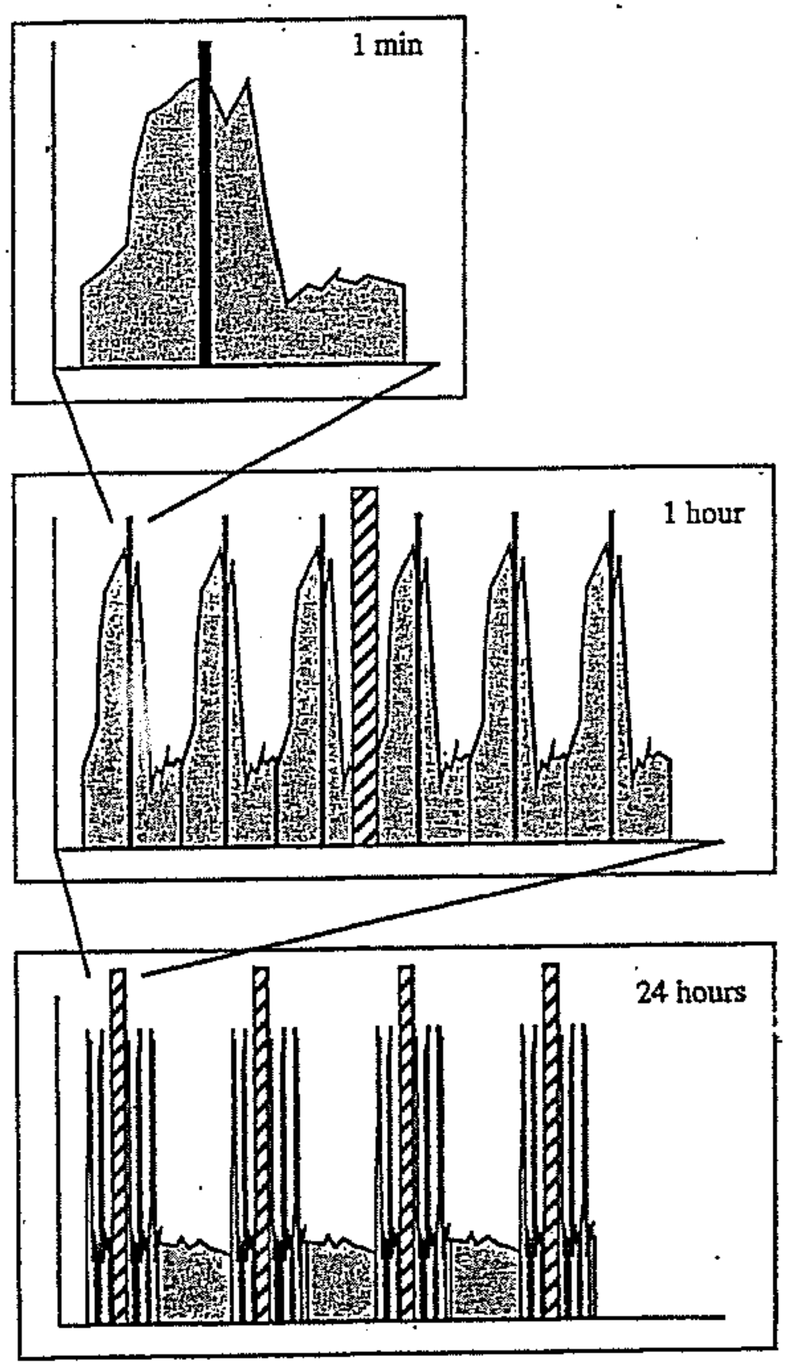
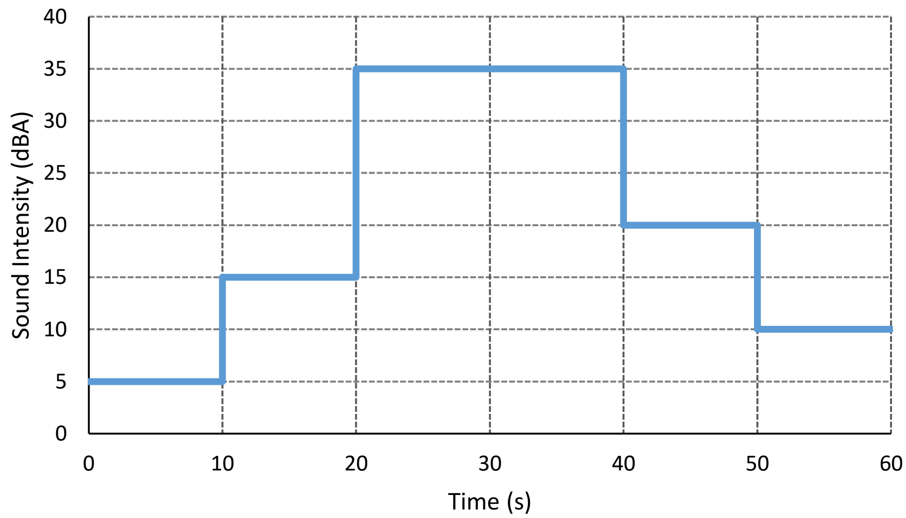


Figure 2.33 Aggregation of the Noise Impacts of Aircraft Operations

Example: SEL & Day Night Equivalent

The following plot represents the noise perceived at a given location produced by one aircraft passing by.



a) Calculate the Sound Exposure Level of a single aircraft.

During day time (6 am – 10 pm), the airport handles 25 operations per hour. During night time (10 pm – 6 am), only 10 operations occur per hour. The noise doses that occur during the night are assigned a surcharge of 10 dBA.

b) Calculate the one hour SEL for both the night and day periods.

c) Calculate the Day-Night Equivalent Sound Level of the airport. Is it acceptable for residential area?

Figure 2.34 illustrates the 85 SEL contours for different jet aircraft types at the beginning of a take-off and demonstrates the dramatic reduction in noise that has occurred with the newer (Chapter 3 of US FAA requirements) aircraft which use state-of-art noise suppression technology. The B727 was introduced in the 1970's and the geographic extent of its noise impacts is illustrated very clearly.

Figure 2.35 illustrates the range of acceptable L_{dn} magnitudes for residential areas where maximum acceptable noise levels are 60 to 65 dBA.

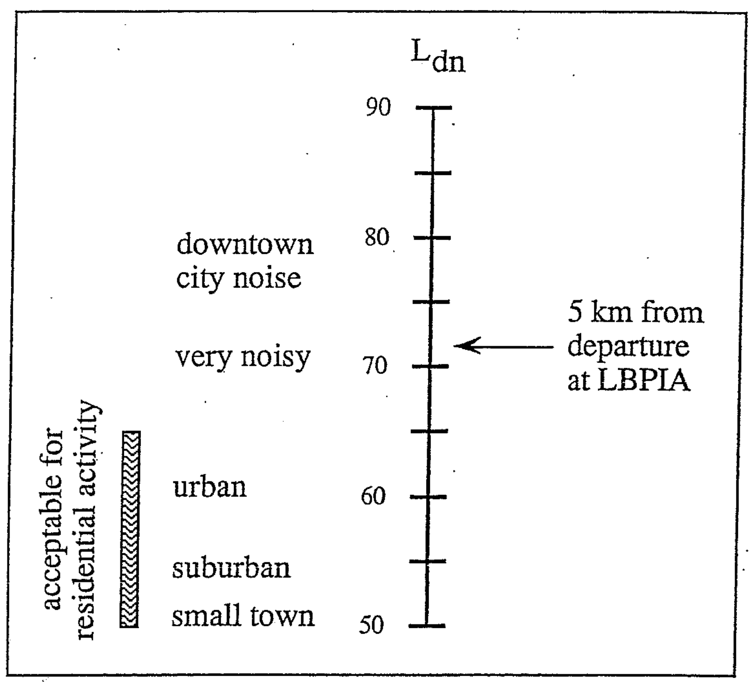


Figure 2.34 Acceptable L_{dn} Levels for Residential Areas

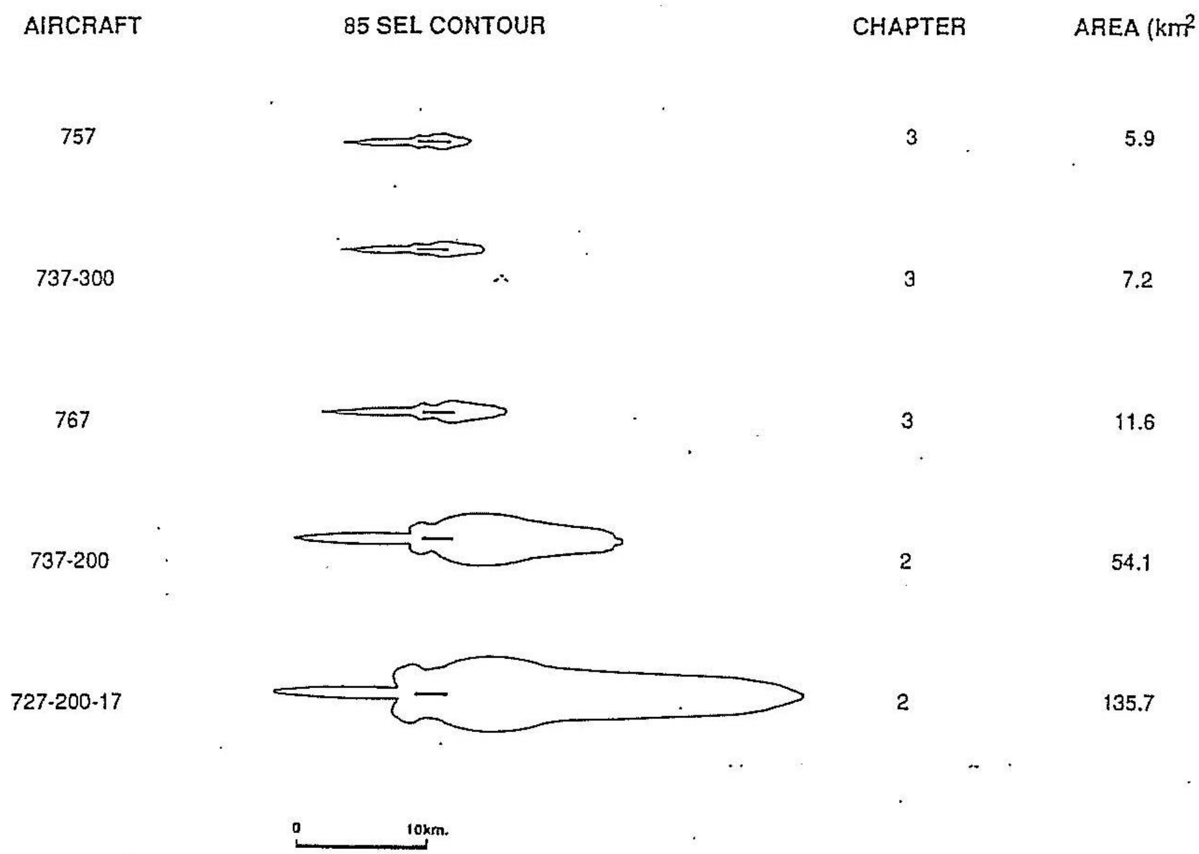


Figure 2.35 85 SEL Noise Contours for Various Jet Aircraft

2.10.4. Noise Exposure Forecast (NEF)

The Noise Exposure Forecast (NEF) is an earlier method for estimating the noise impacts of airport operations. It differs from the SEL/L_{dn} approach in the way in which it aggregates the noise generated by the individual airport operations, however there is a general relationship between the two:

Equation 2-4

NEF magnitudes are still used by Transport Canada to delimit the area surrounding airports that are impacted by noise. Figure 2.36 illustrates the 1988 NEF contours for operations at LBPIA in Toronto. It shows the very large area impacted by unacceptable noise levels (> 25 NEF), which is an area of about 200 km^2. The area of unacceptable impact is expected to decrease, even with increasing traffic volumes, as the quieter aircraft form a larger proportion of the commercial fleet.

Figure 2.36 1988 NEF Contours for LBPIA

References:

[Boeing 07] Boeing Commercial Aircraft – Design Groups/Codes (FAA & ICAO) http://www.boeing.com/commercial/airports/faqs/aircraftdesigngroup.pdf

[Braaksma 00] Braaksma, J. and D. Bell, “Customer Service Levels at Canadian Airports”, Transportation Research Board, CD-ROM, Washington D.C., 2000.

[CAPTG 00] Canadian Airfield Pavement Technical Group website www.captg.org.

[CATSA 06] CATSA Act Review Secretariat, “Flight Plan: Managing the Risks in Aviation Security, Report to the Advisory Panel”, Her Majesty the Queen in Right of Canada, represented by the Minister of Transport, Ottawa, Ontario, 2006.

[CATSA 07] Canadian Air Transport Security Authority website www.catsa-acsta.gc.ca

[Graff 10] Graff, P. “The challenge to increase our handling capacity has been met”, International Airport Review, Issue 6, 2010.

[GTAA 08] Greater Toronto Airport Authority, “Taking Flight: The Airport Master Plan 2008-2030”

[GTAA 16] Greater Toronto Airport Authority website http://www.torontopearson.com/

[Humphreys 00] Humphreys, I., and G. Francis, “A Critical Perspective on Traditional Airport Performance Indicators”, Transportation Research Board, Washington D.C., 2000.

[ICAO 96] International Civil Aviation Organization, “Initiatives Concerned With Large Impact on Airport Operations of Future Large aircraft”, International Civil Aviation Organization, V. 51, N. 7, P. 12, 1996.

[ICAO 2005] International Civil Aviation Organization, “Aerodrome Design Manual”.

[Tighe 00] Tighe, S.L. “The Technical Considerations for Canadian Airfield Pavements”, University of Waterloo, February 2000.

[TC 10] Transport Canada Website https://www.tc.gc.ca

[TC 12] Transport Canada Website https://www.tc.gc.ca/eng/civilaviation/opssvs/secretariat-terminology-glossary-814.htm

3. Highway Planning and Design

CHAPTER THREE

3.1 Introduction

Civil Engineers are concerned with four major aspects of the design of road networks and these are:

1. the overall location of the highway route, where the objective is to minimize environmental impacts and costs (land purchase and construction costs),
2. the geometric design of the road so that it is capable of handing the vehicles travelling at a specific design speed,
3. the ability to adequately carry the required traffic capacity on each link of the road network, and
4. the design of pavements and bridges to support the expected traffic volumes and wheel loads.

3.2 Road Classification

Highway systems provide two fundamental functions, vehicle movement between points of origin and destination and land access. These two functions are in conflict as roads which provide direct access to residential and commercial activities tend to be disruptive to traffic flow as vehicles slow to enter access points (driveways, parking lots, etc.) or accelerate to enter the roadway. Higher traffic flow efficiencies and lower accident rates are found on controlled access highways.

A simple but very important element of road network design is the functional roadway classification. Road classification schemes establish hierarchies of road types based on the extent to which each road provides vehicle movement and land access. The five major road classifications are as follows:

Local roads provide direct access to abutting land uses while freeways provide no direct access to land. Motorists must exit freeways by interchanges and gain access to land from lower classification levels.

Roadways are further classified based on their urban or rural nature, the type of median present (i.e. divided or undivided) and their design speed. Rural and urban classifications refer to the predominate adjacent land use. Divided roads limit the number of opposing vehicle conflict points to controlled locations. The design speed of a roadway is the speed selected for the purposes of design of geometric features and is discussed in more detail later in this chapter.

Figure 3.1 illustrates the differences between arterial, collector, and local streets and roads for urban and rural networks.

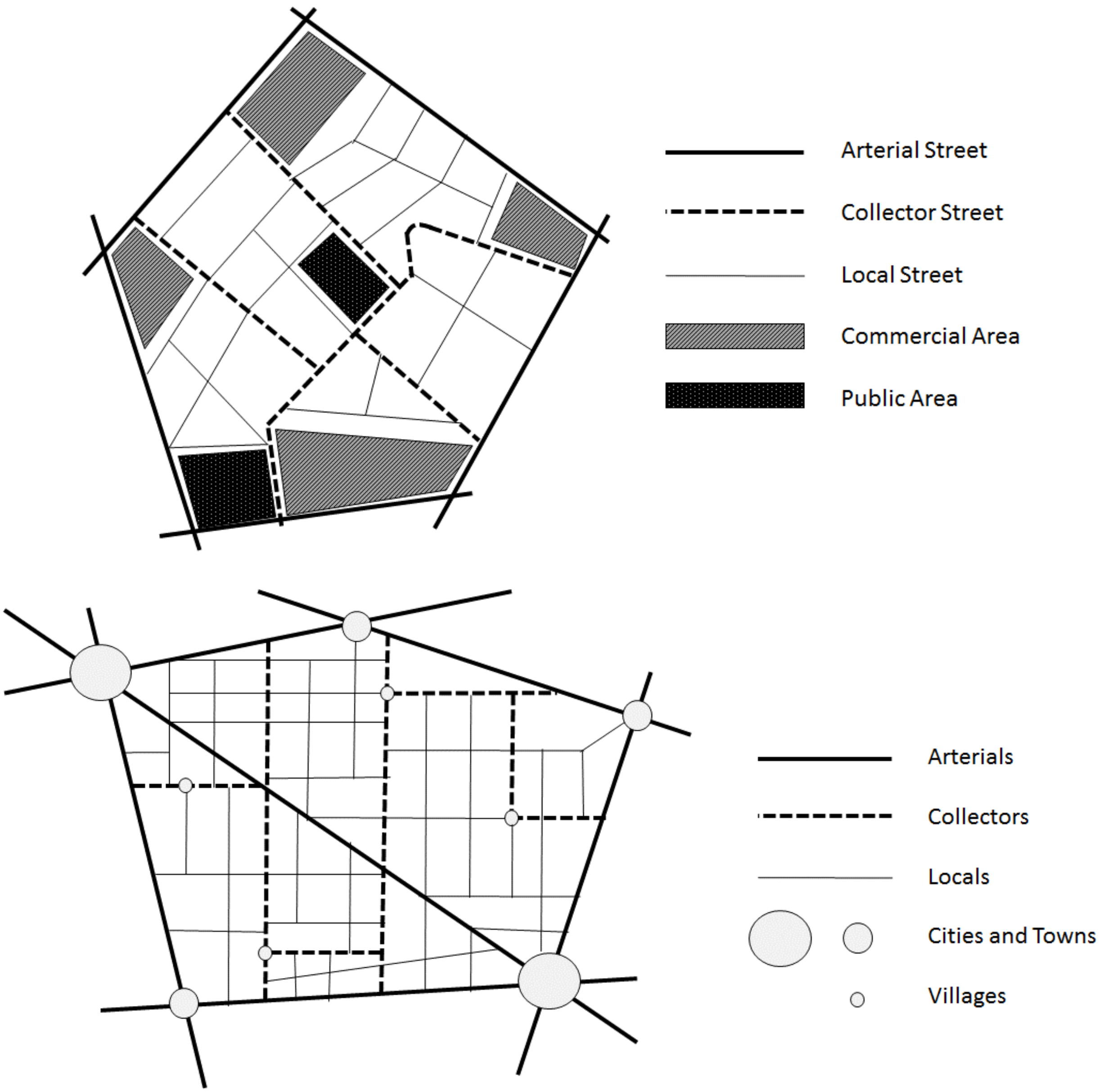


Figure 3.1 Urban (top) and rural (bottom) road classifications

Examples of each road type within the City of Waterloo are illustrated in Figure 3.2. For each of the following roadways indicate whether it is an is an example of a freeway/expressway (A), an urban arterial route (B), collector street (C), or local street (D).

- Highway 85, or the Conestoga Parkway
- King Street
- Northfield Drive/Westmount Road
- Glen Forrest Boulevard
- Albert Street
- Streets bounded by Glen Forrest, Weber, and Bearinger

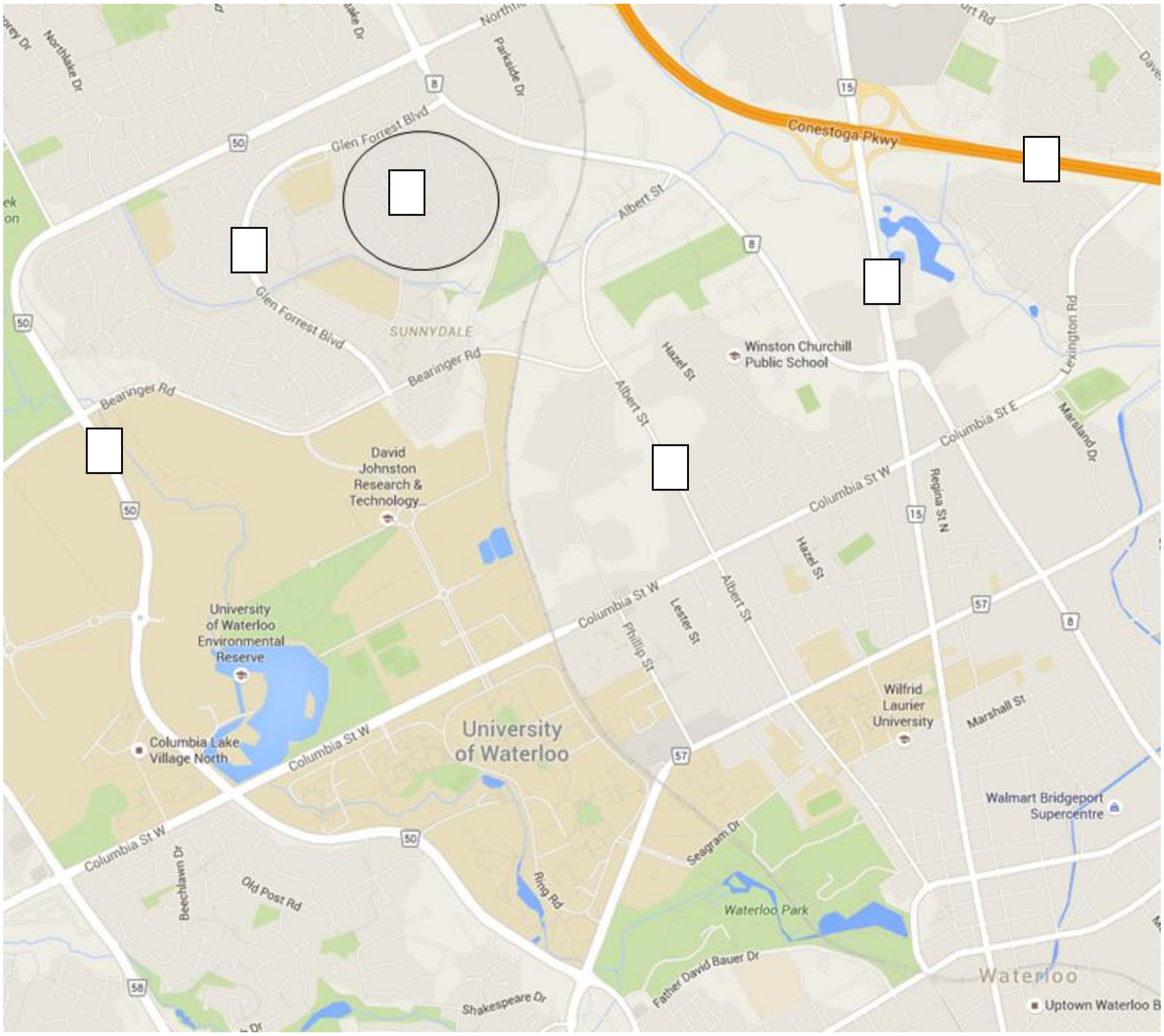


Figure 3.2 Road classifications in Waterloo (Google Imagery: Google, CNES 2016)

Table 3.1 lists the codes used to identify various highway classification groups. A RAD 100 roadway is identified as a (R)ural (A)rterial (D)ivided roadway with a design speed of 100 km/h.

The characteristics of rural road classifications are shown in Table 3.2Table 3.2 Characteristics of Rural Road Classifications

and urban road classifications in Table 3.3.

Table 3.1 Roadway Functional Classification Codes [TAC 2013]

	Design Speed (km/h)	Classification				
		Local	**Collector**	**Arterial**	**Expressway**	**Freeway**
Rural	50	RLU50				
	60	RLU60	RCU60			
	70	RLU70	RCU70			
			RCD70			
	80	RLU80	RCU80	RAU80		
			RCD80	RAD80		
	90	RLU90	RCU90	RAU90		
			RCD90	RAD90		
	100	RLU100	RCU100	RAU100		
			RCD100	RAD100		RFD100
	110	RLU110	RCU110	RAU110		
			RCD110	RAD110		RFD110
	120			RAU120		
				RAD120		RFD120
	130			RAU130		
				RAD130		RFD130
Urban	30	ULU30				
	40	ULU40				
	50	ULU50	UCU50	UAU50		
			UCD50			
	60		UCU60	UAU60		
			UCD60	UAD60		
	70		UCU70	UAU70		
			UCD70	UAD70		
	80		UCU80	UAU80		
			UCD80	UAD80	UED80	UFD80
	90			UAD90	UED90	UFD90
	100			UAD100	UED100	UFD100
	110				UED110	UFD110
	120					UFD120

Table 3.2 Characteristics of Rural Road Classifications

FUNCTIONAL CLASSIFICATION	RURAL FREEWAYS	RURAL ARTERIALS	RURAL COLLECTORS	RURAL LOCALS
Traffic Service	Optimum mobility	Traffic movement primary consideration	Traffic movement & land access equal importace	Traffic movement secondary consideration
Land Service	No access	Land access secondary consideration	Traffic movement & land access equal importance	Land access primary consideration
Range of Traffic Volume A.A.D.T	More than 10,000	1,000 - 20,000	200 - 10,000	Not applicable
Traffic Flow	Free flow	Uninterrupted flow except at signals	Interrupted flow	Interrupted flow
Design Speed	100 - 200 km/h	80 - 110 km/h	60 - 100 km/h	60 - 80 km/hr
Average Running Speed Off-peak Conditions	80 - 120 km/h	60 - 100 km/h	60 - 90 km/h	50 - 80 km/h
Vehicle Type	All types heavy trucks average 20 - 30%	All types up to 20% trucks	All types up to 30% trucks mostly single unit type	Predominantly passenger cars and light to medium trucks and occational heavy trucks
Percentage of Total Length	Up to 5	5 - 10	10 - 20	75 approx.
Connect to	Freeways Arterials Collectors	All Classifications	All Classifications	Arterials Collectors Local

Table 3.3 Characteristics of Urban Road Classifications

FUNCTIONAL CLASSIFICATION	URBAN FREEWAYS	URBAN ARTERIALS	URBAN COLLECTORS	URBAN LOCALS
Traffic Service	Optimum mobility	Traffic movement primary consideration	Traffic movement & land access equal importace	Traffic movement secondary consideration
Land Service	No access	Land access secondary consideration	Traffic movement & land access equal importance	Land access primary consideration
Range of Traffic Volume A.A.D.T	More than 75,000	5,000 - 50,000	1,000 - 20,000	Not applicable
Traffic Flow	Free flow	Uninterrupted flow except at signals and cross walks	Interrupted flow	Interrupted flow
Design Speed	80 - 120 km/h	80 - 110 km/h	60 - 90 km/h	60 - 80 km/hr
Average Running Speed Off-peak Conditions	80 - 110 km/h	50 - 90 km/h	40 - 70 km/h	40 - 60 km/h
Vehicle Type	All types up to 20 trucks	All types up to 20% trucks	All types	Passenger and service vehicles
Percentage of Total Length	Up to 10	Up to 30	Up to 30	70 approx.
Connect to	Freeways Arterials	Freeways Arterials Collectors	Arterials Collectors Locals	Collectors Local

3.3 Traffic Volume Characteristics

Traffic volumes vary throughout the day, the week and the year. Typically, traffic increases during morning and evening rush hours, urban traffic increases during the week, rural-recreational traffic increases during the weekend, and traffic increases during the summer months especially for rural-recreational traffic.

The general unit of measure for traffic volume is Average Daily Traffic (ADT). This is the total two-way traffic volume passing a point on the roadway in a given day. Since daily traffic varies through the week and year, it is often more appropriate to consider the Average Annual Daily Traffic (AADT), which incorporates the daily variations throughout the year. Other daily traffic volume data includes:

These values can be determined by applying the appropriate adjustment factors to a daily count or through an extensive traffic monitoring program.

Design Hourly Volume (DHV) is used to address the hourly and peak conditions needed to address many design requirements. Design Hourly Volume is the volume during 30th highest hour of the year. Due to the constraints of obtaining hourly traffic volumes throughout the year, the Design Hourly Volume is typically determined as a percentage of AADT based on adjustment factors. Figure 3.3 provides an estimate of hourly traffic volumes as a percentage of AADT plotted against their rank order in the year for various roadway types. Recreational routes tend to have even higher variability in hourly traffic flows with the DHVs of approximately 40 percent of AADT. Peak Hourly Factor (PHF) is a term used to represent peaking within the design hour. For the purposes of this course, we will use Figure 3.3 for determining DHV, but this value should be measured on site for design projects.

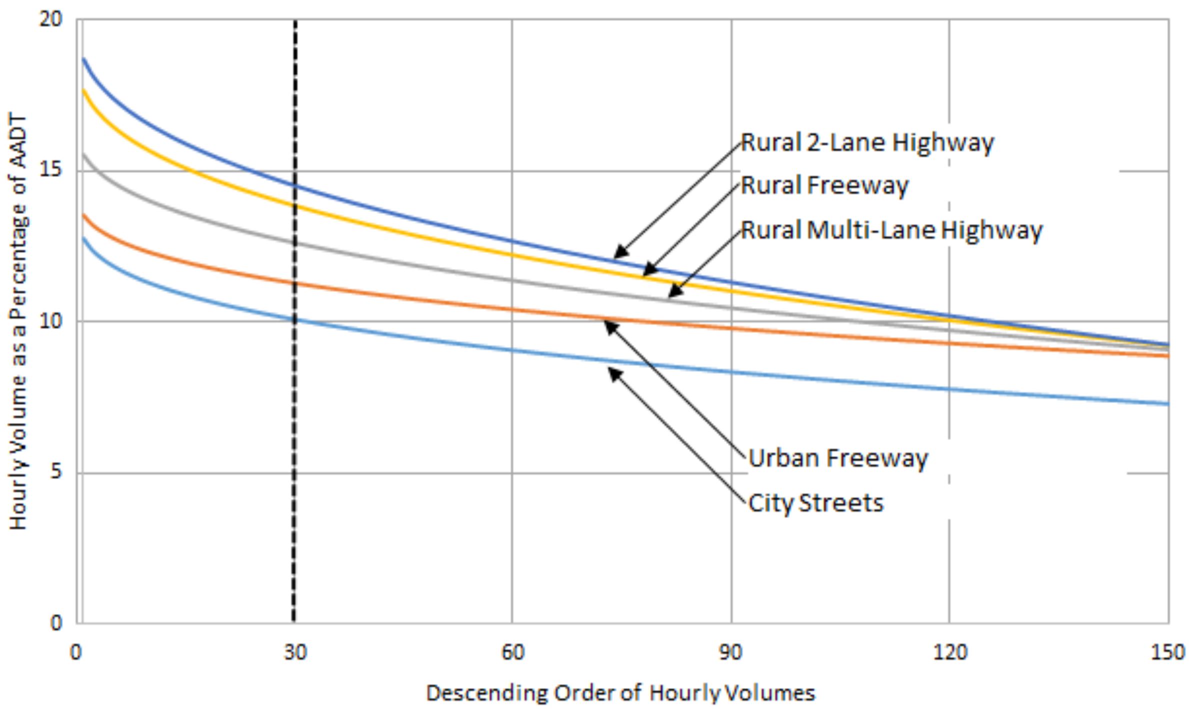


Figure 3.3 Hourly Traffic Volumes as % AADT vs. Rank Order in Year

Another consideration when examining traffic volumes is the directional split of traffic. Traffic flow on two-way facilities is generally equal for daily periods but can be unbalanced during peak periods. This is particularly true for roads connecting suburbs and city centres.

Traffic characteristics must also consider the composition of traffic. Vehicles are often separated into passenger vehicles, trucks and buses. The composition of trucks and buses can vary widely among roadways and should be determined through field evaluations.

Traffic monitoring can provide useful information about the existing traffic conditions on a roadway. When examining new roadways and improvements to existing highways it is necessary to consider both the current and future traffic requirements. Traffic projections are typically made for a period of 10 to 20 years. Traffic growth can occur from several sources, including a general trend towards increased vehicle use, diversion of traffic from other routes because of the improvements (usually occurring within a year or two of the improvement/new highway), and future development in the surrounding area.

3.4 Design Speed

The design speed of a highway is the speed selected for design and determines the geometric characteristics and quality of the highway. It is the maximum safe speed that can be maintained when conditions are favourable that the design features of a highway govern. It must be understood that the design speed of a highway is not safe for severe weather conditions such as ice or snow cover or for high traffic conditions where there is interaction between vehicles. The design speed of a highway is dependent on the functional classification and individual constraints of the highway.

The selected design speed determines various geometric requirements including minimum sight distance, horizontal and vertical alignment, superelevation, shoulder and lane widths, lateral clearances and access control requirements. Generally, design speeds are 20 km/h greater than the posted speed. For minor highways with low traffic, rugged terrain or environmental constraints it may be appropriate to reduce the design speed to that of the posted speed.

Although the selected design speed provides the minimum design standard for a section of highway, designers should understand that the design speed is a minimum value, and higher design requirements should be incorporated where practical. Where designers are restricted from incorporating the required design standard, it is important to warn drivers in advance of the lower speed conditions. Signs such as those shown in Figure 3.4 can be used to accomplish this.

Figure 3.4 Signs warning of road sections with lower design speed

3.5 Stopping Sight Distance

Stopping sight distance is the roadway distance between a vehicle and an object in the vehicle's path. Minimum stopping sight distance is the minimum distance required to bring a standard vehicle to a stop under favourable conditions. Standards for minimum stopping sight distance involve a perception and reaction time (distance) and braking distance based on various coefficients of friction for wet pavement conditions. Typically, a conservative perception and reaction time of 2.5 seconds is used for stopping sight distance calculations. The distance travelled during the perception and reaction time is typically the design speed multiplied by the perception and reaction time, although some agencies such as MTO use an assumed speed, which is less than the design speed for design speeds above 70 km/h. The difference between assumed speed and design speed represents recognition that under wet conditions most drivers will drive less than the design speed for higher design speeds.

Equation 3-1: Perception/Reaction Distance

where:

Braking distance varies greatly from vehicle to vehicle but is determined based on the following generalized equation.

Equation 3-2: Breaking Distance

where:

Figure 3.5 shows some typical coefficients of tire-pavement friction for various conditions. A typical value for wet pavements used in the derivation of design standards is 0.3. This is representative of worn tires on pavements with poor surface conditions.

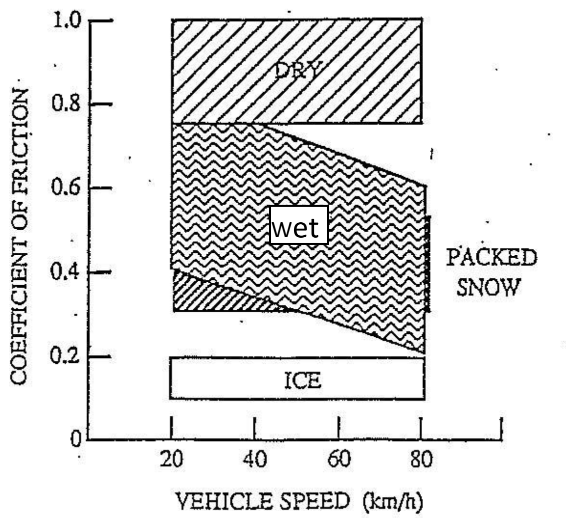


Figure 3.5 Typical Coefficients of Friction

Example: Stopping Sight Distance (SSD)

The stopping sight distance for a design speed of 80 km/h over level terrain would be calculated as follows (note that a friction value of 0.31 based on design reference documents):

3.6 Vertical Alignment

The vertical alignment defines the profile of the road along its centreline. The vertical alignment of a road consists of a series of tangent sections of various slopes connected by a series of vertical curves (either crest curves or sag curves) of parabolic form. A typical vertical alignment is illustrated below in Figure 3.6. Vertical alignments are typically expressed with K-values to define both crest and sag curves and grade percentages to define the tangents. The larger the K value the gentler the curve.

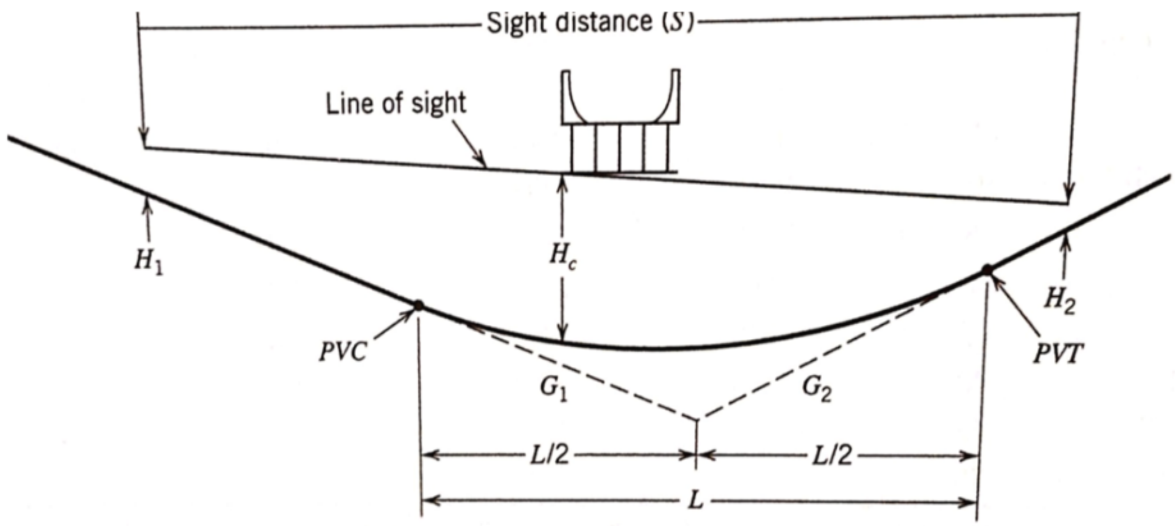


Figure 3.6 An Example of a Vertical Road Alignment

The vertical alignment of a road influences the extent of a driver's vision both ahead and behind. The vertical alignment introduces forces, which influence vehicle speeds, driver comfort and the ability to accelerate and decelerate. The profile of the road also impacts the construction costs of a highway as it influences the cut and fill quantities.

3.6.1. Tangent Grades

The grade of any highway impacts the safety and operation of a highway since it affects drainage and vehicle performance. Grades impact vehicle performance through deceleration of (heavy) vehicles, decreased vehicle deceleration on downhill grades, the ability of trucks to negotiate icy grades and increased fuel consumption on uphill grades. Selection of a (maximum) gradient typically considers design speed, highway classification, traffic volumes, traffic composition, terrain, property requirements, and environmental constraints. On grades of 3% or less, passenger car operation is slightly affected, and trucks are affected primarily on long grades only. On grades of 5%, car operation is generally slightly affected but trucks experience a significant loss of speed and may have additional difficulty under icy conditions. Steep grades can cause maintenance problems because of the relatively rapid water movement leaving the pavement.

3.6.2. Vertical Curves

Tangents are connected using parabolic vertical curves of the equation shown below. Parabolic curves provide a constant rate of change of slope between tangents.

Equation 3-3

where:

The variable "r" represents the rate of change of grade but engineering practice is to express the severity of a vertical curve in terms of K, the reciprocal of r, where:

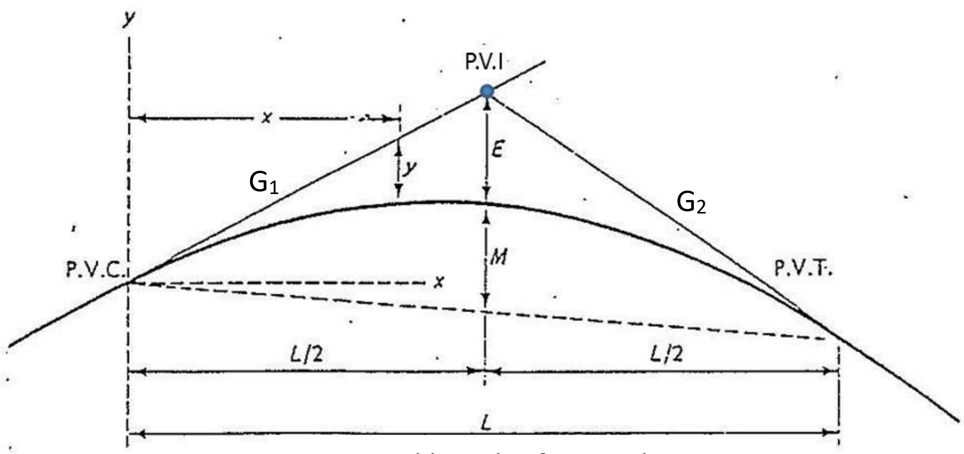
Equation 3-4

where:

The geometry of a vertical curve is shown below in Figure 3.7.

Vertical curves are either a crest curve or a sag curve. Although the geometric characteristics of these two types of curves are the same, the effects of these curves to drivers is very different. Crest curves are a concern for sight distance (Figure 3.8). Sag curves are a concern for headlight visibility (Figure 3.9) as the vehicle stopping distance can exceed the ability to see objects in the headlights. A vertical headlight angle of 1 degree is used for determining headlight visibility. Driver comfort is a concern for both types of curves, but driver comfort is maintained when the sight distance or headlight visibility requirements are met. When illumination is provided, the sag curve requirements can be based on driver comfort. Design K values are provided in Table 3.4 for crest curves and Table 3.5 for sag curves.

The determination of stopping sight distance on vertical crest curves involves assumptions regarding the height of the driver's eye above the roadway and the height of the object above the pavement. For Ontario, a driver's eye is assumed to be 1.05 m high and the object is assumed to be 0.38 m high (the legislated minimum tail-light height). A 1994 study found that more than 99% of Canadian vehicles had a driver's eye height of at least 1.05m, and therefore this is a conservative assumption to use when designing roadways. The object height can be changed depending on the specific circumstances of a given roadway. For instance, if small but dangerous debris is often encountered, such as rocks from adjacent rock slopes, a smaller object height can be considered. Generally, the use of 0.38 m has been found to be conservative for design.



Horizontal length of vertical curve

where:

y	= elevation (m)	r	= rate of change of grade
P.V.T	= Point of Vertical Tangency (m)	G_1	= tangent grade entering
P.V.C	= Point of Vertical Curvature (m)	G_2	= tangent grade departing
x	= Horizontal Distance (m)	P.V.I	= Point of Vertical Intersection (m)

Figure 3.7 Geometric Properties of Parabolic Curve

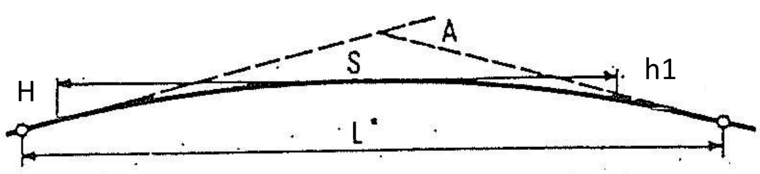


Figure 3.8 Crest Curve

Length of Crest Curve (m)

Equation 3-5: Length of Crest

Equation 3-6: Minimum Crest

Equation 3-7: Minimum Crest

where:

Table 3.4 K Magnitudes for Crest Vertical Curves

Design Speed (km/h)	40	50	60	70	80	90	100	110	120	130
Minimum Crest, K (m)	4	7	15	22	35	55	70	85	105	120

L_{crest} in metres not less than design speed in km/h

Example: Required Length of Crest Curves

A road section has a posted speed limit of 80 km/hr. The crest of a hill is being re-built for safety. The sides of the hill are each 3% (up and down). Using the values in Table 3.4, determine the length of the crest curve that is required.

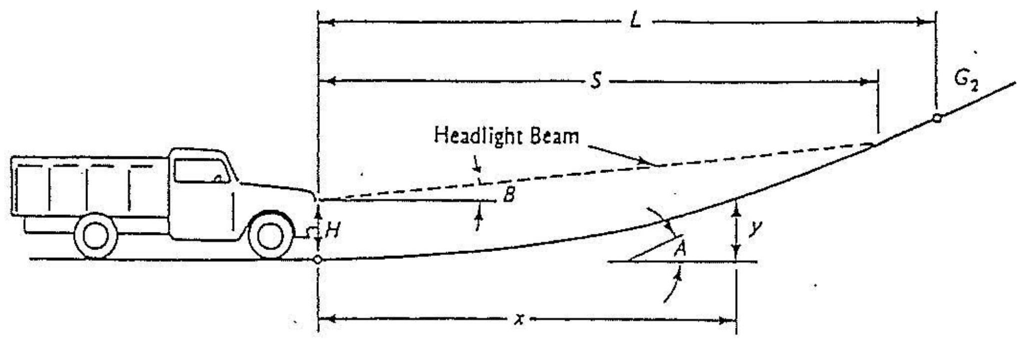


Figure 3.9 Geometry of Headlight Control on Sag Vertical Curves

Length of Sag Curve (m)

Equation 3-8

where:

Table 3.5 K Magnitudes for Sag Vertical Curves

Design Speed (km/h)	Minimum Stopping Sight Distance (m)	Minimum Sag, K headlight control (m)	Minimum Sag, K comfort control (m)
40	45	7	4
50	65	12	6
60	85	18	9
70	110	25	12
80	140	32	16
90	170	40	20
100	205	50	25
110	250	62	30
120	290	73	36
130	330	85	43

L in metres not less than design speed in km/h; centripetal acceleration 0.3 m/s^2

Minimum K-values for sag curves are often useful in urban settings, as the total length of the curve is ideally minimized. This reduces the number of adjacent roadways and services which are affected and thereby reduces the construction costs. The 2014 re-design and reconstruction of Weber St. beside the train station in Kitchener is a good example of this practice.

Example: Sag Curves

A sag curve is being constructed in a rural setting to transition between a horizontal section and a section with a 4% grade. The posted speed limit in this area is 90 km/hr and the design speed 110km/hr has been chosen. The initial design does not include streetlights in this section. The coefficient of friction of the road can be considered to be 0.29 and the headlights are 0.5m off the ground. Using Equation 3-8, what is the length of the curve that is required? Using K values, what is the L? What length would be required if there were street lights on the vertical curve?

3.7 Horizontal Alignment

The horizontal alignment defines the roadway configuration in plan view. The horizontal alignment of a road consists of tangent sections, circular curves and transition curves (also referred to as spiral curves), which connect the tangent sections to the circular curves as shown in Figure 3.10. Circular curves are used to change the direction of travel on a roadway and are designated by their radii in metres.

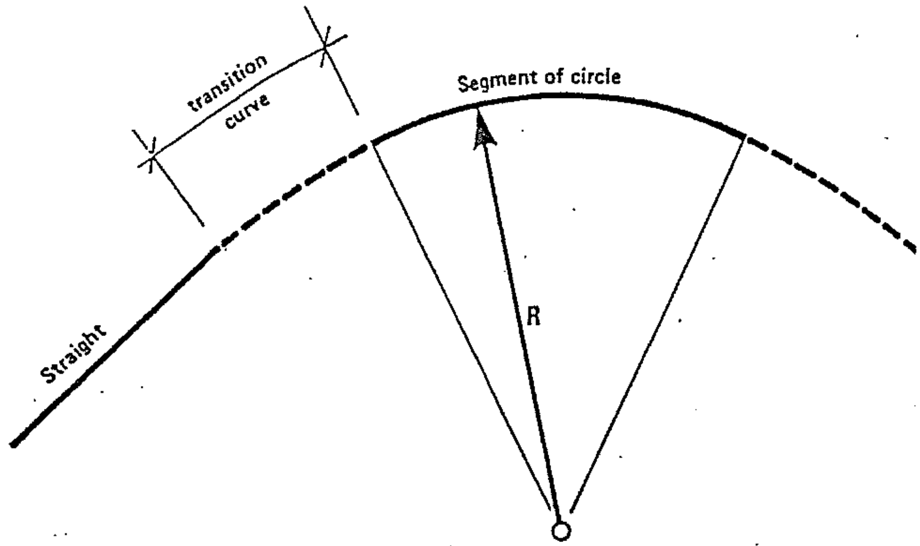


Figure 3.10 Components of Horizontal Alignment

3.7.1. Circular Curves

Minimum horizontal curvature is governed by design speed, pavement friction, and superelevation, as outlined below. The equation is derived based on the forces acting on a vehicle moving around a horizontal curve as shown in Figure 3.11. Centrifugal forces act to create an overturning moment about the outer wheel contact points with the pavement. A stabilizing moment is created by the weight of the vehicle and the frictional forces developed between the tires and the pavement.

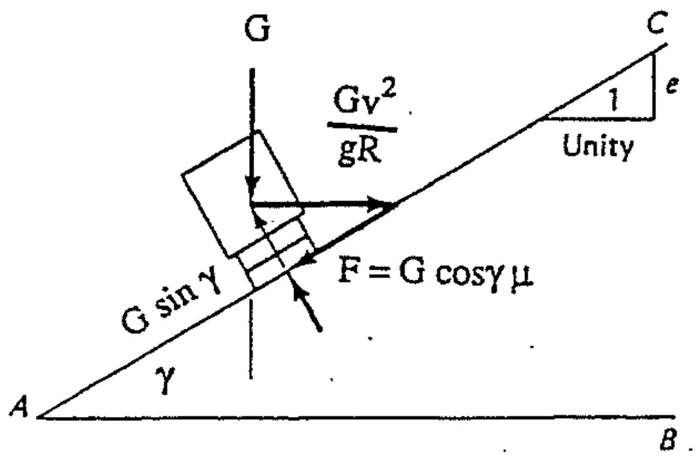


Figure 3.11 Forces acting on vehicle moving around horizontal curve

Equation 3-9: Equilibrium

dividing by $G \cos \gamma$and solving $\tan \gamma = e$

Equation 3-10

Equation 3-11: Circular Curve Radius

where:

Superelevation is the banking of a curve to assist a vehicle as it rounds the curve. A maximum superelevation of 6 percent is used for most applications although some freeways do incorporate superelevation as high as 8 percent. Higher rates of superelevation are not recommended as icy conditions will cause slow moving vehicles to slide inward from the gravitational forces.

3.7.2. Spiral or Transition Curves

Transition curves (also referred to as spiral curves) provide a transition between tangent sections and circular curves and adjacent circular curves of different radii. The transition serves several purposes. It develops superelevation for the circular curve and allows for the driver to steer gradually into and out of a circular curve. Without a transition curve, drivers would experience a sudden increase in radial acceleration, which usually results in unsatisfactory driver behaviour as drivers tend to steer a path with a radius larger than the actual circular curve radius and then over-correct. The spiral transition provides a constant increase in steering angle from 0 to the angle required by the circular curve and eliminates the need for an instantaneous steering response from the driver.

Transition curves are expressed in terms of their spiral parameter, A given by the equation below. The spiral parameter is a measure of the rate of change of the radius of the spiral. Large spiral parameter values indicate a slower the rate of change in radius (and a gentler transition).

Equation 3-12

where:

Minimum spiral parameter values are based on three factors, aesthetics, superelevations, and driver comfort as shown in Figure 3.12.

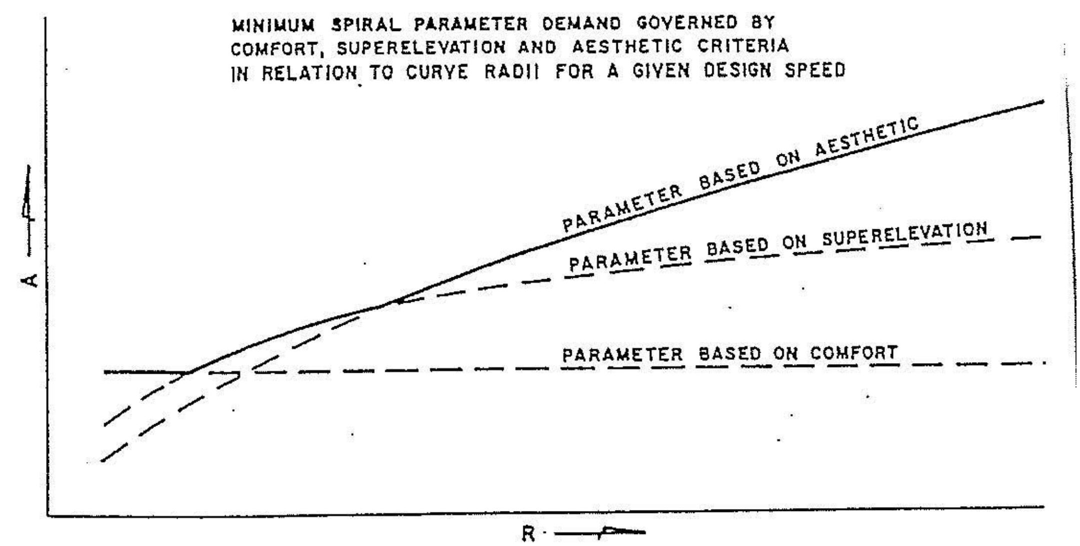


Figure 3.12 Different Criteria for Spiral Lengths for a Design Speed of 100 km/h

3.7.3. Spiral Parameter based on Comfort

Vehicles travelling along a curve experience centripetal acceleration. In a spiral curve the centripetal acceleration increases as a vehicle transitions from a tangent (i.e. a curve with infinite radius) to a circular curve (of lower radius). The reverse occurs when transitioning from a circular curve to a tangent. Too high a rate of change also causes drivers to respond to the curvature in an inappropriate manner and over-steer the curve. The maximum rate of change in acceleration, ΔC, used by the most transportation agencies is 0.6 m/s^3

Equation 3-13

where:

3.7.4. Spiral Parameter based on Superelevation

A second constraint for spiral curves is the minimum length of spiral curve required to develop superelevation in a curve. The minimum length of spiral curve required to develop full superelevation in the circular curve is given by:

Equation 3-14

where:

The relative slope is the slope of the outer edge of the pavement in relation to the profile control line (usually the centre line of the road). The maximum relative slope is a function of the design speed and decreases with increasing speed to minimize the effects on drivers of the longitudinal increase in the elevation of the outside edge of the pavement. The recommended values are:

Table 3.6 Relative Slope Values

design speed (km/h)	60	80	90	100	110	120	130
relative slope (m/m)	0.0060	0.0051	0.0047	0.0044	0.0041	0.0038	0.0036

The A-parameter magnitude that satisfies this constraint may be calculated from the spiral length calculated in Equation 3.12 and 3.14 For example, at a design speed of 90 km/h and with e = 0.06 m/m and w = 15 m (4 lanes), the length of the spiral required for proper superelevation development would be 15 x 0.06 / (2 x 0.0047) = 95.745 m. This means an A-parameter magnitude of $(95.745 \times 340)^{0.5}$ = 180 m. The spiral length, and A-magnitude for minimum circular curve radius, are larger than for comfort control.

3.7.5. Spiral Parameter based on Aesthetics

Short spiral curves can be visually unappealing from a driver's perspective. This can result in unexpected behaviours from drivers travelling on the curves. In general, the length of the spiral should be such that the driving time on the spiral is at least equal to 2 seconds. The appropriate A-magnitude is calculated as follows:

Equation 3-15

Equation 3-16

Example: Required Length of Spiral Transition Curves

A curve is being constructed in an area where the design traffic speed is 100 km/hr. If the superelevation of the curve is to be 5%, determine the minimum radius of the curve. Based on that radius, determine the required length of the spiral transition curves. Assume that the design side friction value is 0.13 and there are 4 undivided lanes.

3.8 The Clear Zone Concept

Highway engineers must recognize that circumstances will arise to cause vehicles to leave the roadway. Hazards located near the roadway pose a risk of being hit by a vehicle leaving the roadway. To provide safety for vehicles that may leave the highway, designers should provide a clear zone, which provides a reasonable area for a vehicle to recover should it leave the roadway. This reasonable area is based on a number of factors but should not be considered to be an absolute or precise value. Figure 3.13 and Figure 3.14 show the probability distribution for distance from the edge of pavement that a vehicle can travel for an operating speed of 100 km/h. Clear zone is defined as the distance from the edge of the travelled portion of the roadway to a hazard. The minimum required clear zone is influenced by a number of factors including design speed, traffic volumes, the presence of cut or fill slopes, the steepness of slopes, and horizontal curve adjustments. Standard clear zone distances and horizontal curve adjustments are provided in Table 3.7 and Table 3.8.

Figure 3.15 illustrates what is meant by cut and fill slopes on a typical roadway cross section.

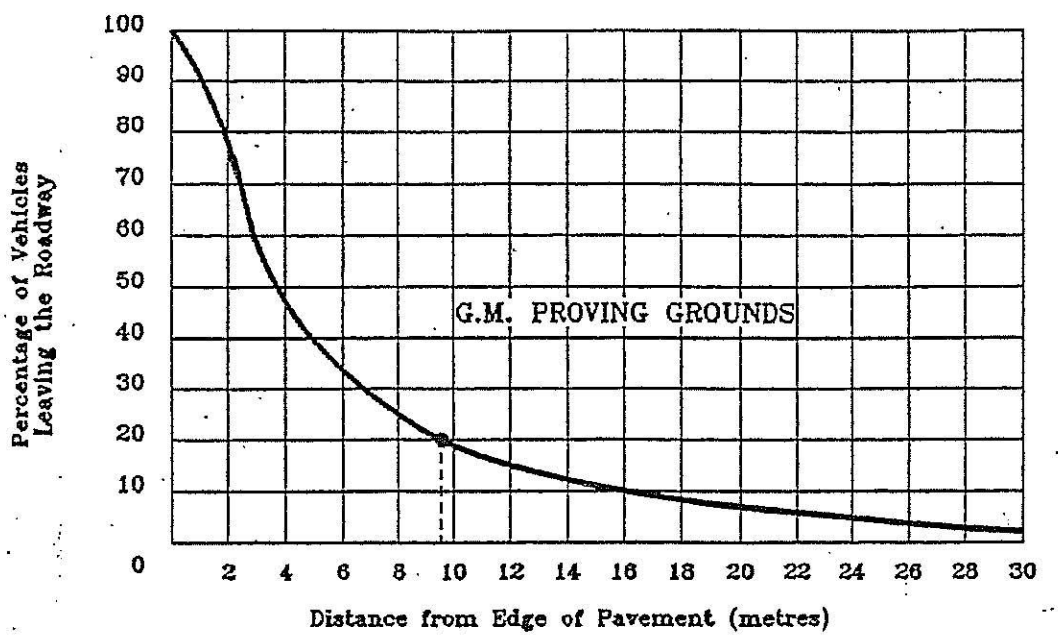


Figure 3.13 Depth of Penetration from Edge of Pavement for Errant Vehicles

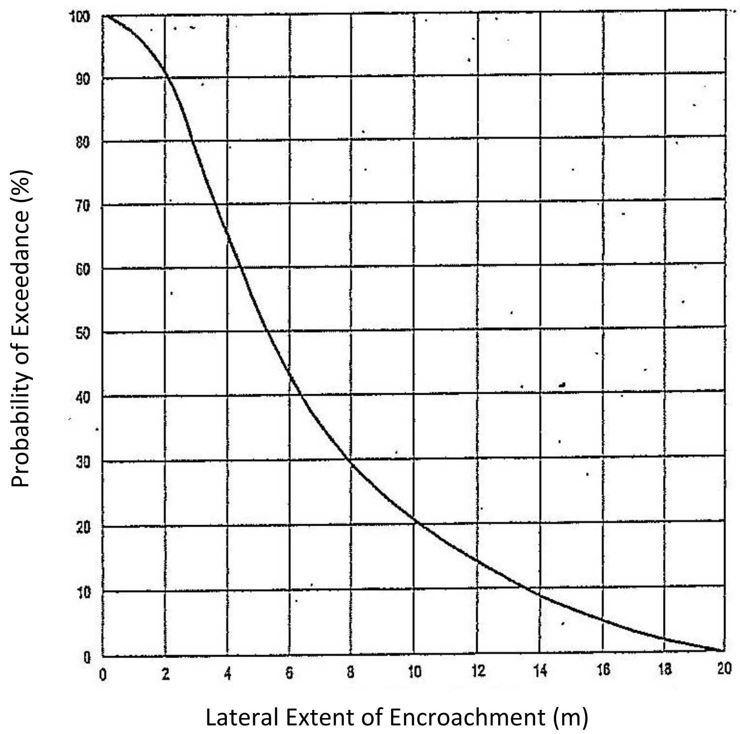


Figure 3.14 Clear Zone Encroachment Probability

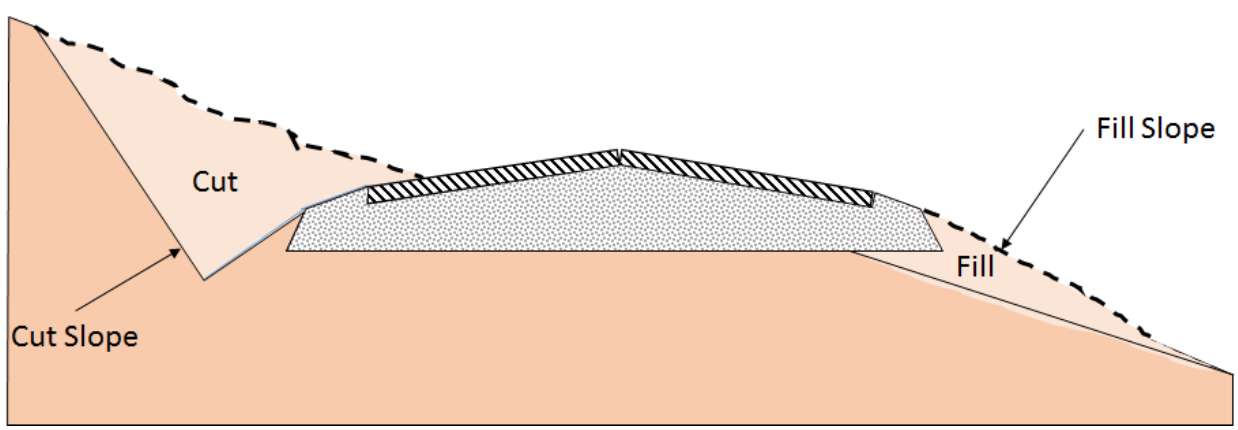


Figure 3.15 Cut and Fill Slopes Illustration

Table 3.7 Clear Zone Distances (m)

Design Speed (km/hr)	Design ADT	Fill Slopes			Cut Slopes		
		6:1 or flatter	5:1 to 4:1	3:1	3:1	5:1 to 4:1	6:1 or flatter
≤ 60	< 750	2.0 - 3.0	2.0 - 3.0	see note	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
	750 - 1499	3.0 - 3.5	3.5 - 4.5	see note	3.0 - 3.5	3.0 - 3.5	3.0 - 3.5
	1500 - 5999	3.5 - 4.5	4.5 - 5.0	see note	3.5 - 4.5	3.5 - 4.5	3.5 - 4.5
	≥ 6000	4.5 - 5.0	5.0 - 5.5	see note	4.5 - 5.0	4.5 - 5.0	4.5 - 5.0
70 - 80	< 750	3.0 - 3.5	3.5 - 4.5	see note	2.5 - 3.0	2.5 - 3.0	3.0 - 3.5
	750 - 1499	4.5 - 5.0	5.0 - 6.0	see note	3.0 - 3.5	3.5 - 4.5	4.5 - 5.0
	1500 - 5999	5.0 - 5.5	6.0 - 8.0	see note	3.5 - 4.5	4.5 - 5.0	5.0 - 5.5
	≥ 6000	6.0 - 6.5	7.5 - 8.5	see note	4.5 - 5.0	5.5 - 6.0	6.0 - 6.5
90	< 750	3.5 - 4.5	4.5 - 5.5	see note	2.5 - 3.0	3.0 - 3.5	3.0 - 3.5
	750 - 1499	5.0 - 5.5	6.0 - 7.5	see note	3.0 - 3.5	4.5 - 5.0	5.0 - 5.5
	1500 - 5999	6.0 - 6.5	7.5 - 9.0	see note	4.5 - 5.0	5.0 - 5.5	6.0 - 6.5
	≥ 6000	6.5 - 7.5	8.0 - 10.0	see note	5.0 - 5.5	6.0 - 6.5	6.5 - 7.5
100	< 750	5.0 - 5.5	6.0 - 7.5	see note	3.0 - 3.5	3.5 - 4.5	4.5 - 5.0
	750 - 1499	6.0 - 7.5	8.0 - 10.0	see note	3.5 - 4.5	5.0 - 5.5	6.0 - 6.5
	1500 - 5999	8.0 - 9.0	10.0 - 12.0	see note	4.5 - 5.5	5.5 - 6.5	7.5 - 8.0
	≥ 6000	9.0 - 10.0	11.0 - 13.5	see note	6.0 - 6.5	7.5 - 8.0	8.0 - 8.5
≥ 110	< 750	5.5 - 6.0	6.0 - 8.0	see note	3.0 - 3.5	4.5 - 5.0	4.5 - 5.0
	750 - 1499	7.5 - 8.0	8.5 - 11.0	see note	3.5 - 5.0	5.5 - 6.0	6.0 - 6.5
	1500 - 5999	8.5 - 10.0	10.5 - 13.0	see note	5.0 - 6.0	6.5 - 7.5	8.0 - 8.5
	≥ 6000	9.0 - 10.5	11.5 - 14.0	see note	6.5 - 7.5	8.0 - 9.0	8.5 - 9.0

Note: Since recovery is less likely on the unshielded, traversable 3:1 slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that travel beyond the edge of the shoulder may be expected to occur beyond the toe of the slope.

Table 3.8 Horizontal Curve Adjustments for Clear Zone Distances

Radius (m)	Design Speed 60	70	80	90	100	110+
900	1.1	1.1	1.1	1.2	1.2	1.2
700	1.1	1.1	1.2	1.2	1.2	1.3
600	1.1	1.2	1.2	1.2	1.3	1.4
500	1.1	1.2	1.2	1.3	1.3	1.4
450	1.2	1.2	1.3	1.3	1.4	1.5
400	1.2	1.2	1.3	1.3	1.4	
350	1.2	1.2	1.3	1.4	1.5	
300	1.2	1.3	1.4	1.5	1.5	
250	1.3	1.3	1.4	1.5		
200	1.3	1.4	1.5			
150	1.4	1.5				
100	1.5					

Note: The clear zone horizontal curve adjustment factor is applied to the outside of curves only. Curve flattor than 900 m do not require an adjusted clear zone.

In using the clear zone values the designer should recognize the offsets provide protection for approximately 80 percent of errant vehicles. The application of clear zone requirements requires judgement. Designers should consider increasing the clear zone where the cost implications are low and in high risk locations. In addition, there is little improvement to safety by removing a tree just inside the clear zone and leaving a forest of trees just outside the clear zone.

Where a hazard exists within a clear zone, the preferred method of treatment is removal. Where a hazard cannot be practically removed from the clear zone, it can be dealt with in a number of alternative methods. It can be minimized (e.g. making a culvert inlet transversable or making a sign support a breakaway device). Hazards can also be shielded with barriers or crash cushions which reduce the severity of an impact. When installing barriers or crash cushions, designers should recognize that these protective devices are still hazards (of a lesser severity) and placed closer to the roadway create a greater likelihood of being hit. The proximity of a barrier device to a roadway is influenced by the deflection area required between a barrier and the hazard. This deflection area is determined based on the type of barrier used. Concrete barriers require essentially no deflection area while cable guide rail requires as much as 3 metres.

Example: Outside Clear Zone Distance

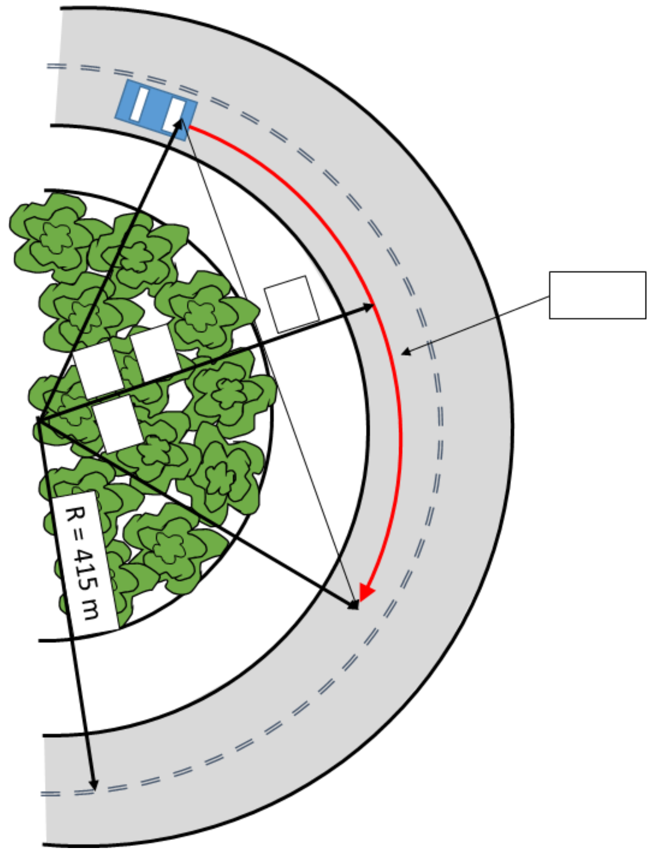
A horizontal curve has a radius of 450 m. The curve is on a section of highway with 1655 ADT with a design speed of 90 km/hr. If the curve is in a 5:1 cut section, what is the recommended outside clear zone distance, based on the above tables? What if the curve is in a 3:1 fill section?

Clear zone must also be considered on the inside edge of curves, but for visibility requirements instead of for vehicle leaving the roadway. A vehicle travelling along the inside lane of a curve will have the available sight distance shortened by obstacles inside the curve, provided the curve is long enough. If the sight distance is reduced below the safe sight stopping distance, then the clear zone on the inside of the curve presents a safety issue.

Example: Clear Zone for Stopping

A highway has a design speed of 110 km/hr. On the highway, there is a curve with a radius of 415 m. On the inside of the curve is a wooded region that is dense enough to block sightlines along the curb. Using SSSD, determine what the required clear zone "m" for the curve is to maintain sufficient distance for stopping.

- f = 0.31
- G = 0%
- Lane width = 3.75 m



3.9 Rural Highway Capacity

The material on rural highway capacity contained in this section is taken from Canadian traffic capacity sources.

3.9.1. Capacity

Capacity is the maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway in one direction (or in both directions on a two-lane or three-lane highway) during a given time period under prevailing roadway and traffic conditions. It is usually given by the service volume at level of service E (see below).

The prevailing conditions may be divided into two general groups and these are (i) prevailing roadway conditions established by the physical features of the roadway and relatively unchanging from day to day; and (ii) prevailing traffic conditions, which are dependent on the nature of traffic on the roadway and thus may change daily or even hourly.

3.9.2. Level of Service

Level of service is a qualitative measure of the effect of a number of factors, which include:

These levels of service are illustrated in Figure 3.16, where the regions of levels of service (A through E) are superimposed on the continuous flow function relating flow rate to volume/capacity ratio (which is a measure of density).

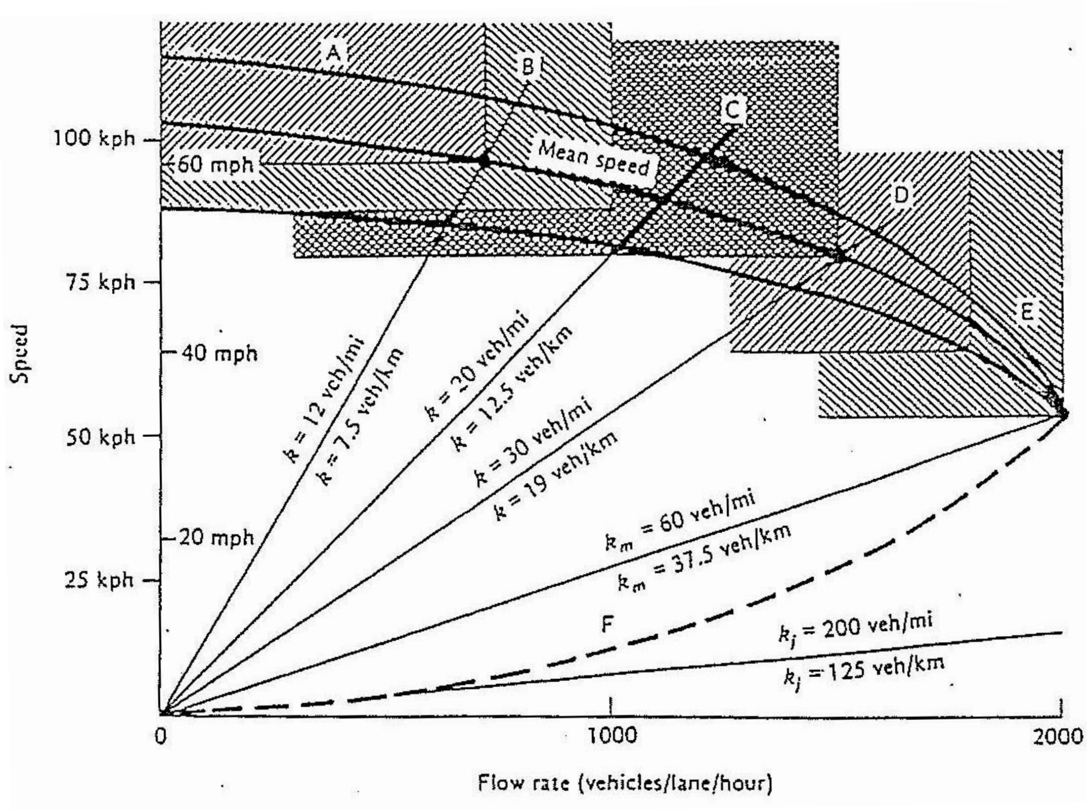


Figure 3.16 Levels of Service for Continuous Traffic Flow Streams Service Volume

Service Volume is the maximum number of vehicles that can pass over a given section of a lane or roadway in one direction on multi-lane highways (or in both directions on a two or three-lane highway) during a specified time period while operating conditions are maintained at a specified level of service. In the absence of a time modifier, service volume is an hourly volume.

3.10 Multi-Lane Rural Highways

Multi-lane rural highways, which are those with four or more lanes, may or may not have full control of access and may be divided or undivided. Divided multi-lane rural highways, including freeways, are analyzed in accordance with the procedures described for freeways. The service volume of all multi-lane highways is given by:

Equation 3-17

where:

The “2000” in Equation 3-17 represents the maximum traffic flow that may be expected from one-lane of a multi-lane road under ideal conditions. The v/c term reflects the vehicle density that is used for a particular level of service, where these are described in Table 3.9. The T and the W adjust the ideal flow for the presence of trucks and the effects of width restrictions.

Values for v/c for undivided highways are taken from Table 3.9. The five levels of service are shown in the first column and the third and fourth columns show respectively, the values of speed and volume/capacity ratio that define these levels of service, where these regions have already been illustrated in Figure 3.16. The table shows the v/c ratios separately for AHS’s (average highway speeds or design speeds) of 120, 100, and 80 km/h. The lower design speeds mean more restrictive geometry and lower concentrations of vehicles are required on these lower quality roads in order to maintain the same quality, or level of service, of traffic flow.

Table 3.10 and Table 3.11 contain the information for calculating the truck equivalency factors T. Table 3.10 contains the passenger car equivalencies for trucks on grades of various lengths and magnitudes and various truck concentrations. The passenger car equivalency may be defined as the number of cars that have to be removed from a traffic stream when a truck is added, in order to maintain the same level of service. The passenger car equivalencies for trucks are calculated from their speed reductions on grades and Figure 3.17 shows the relationship from which the equivalencies are calculated. These reflect the speeds of trucks on grades which constrain the speeds of trailing traffic in the same lane to the same speed. The first truck added to a traffic stream has the largest impact and the marginal impacts on capacity of additional trucks is less, reducing the average passenger car equivalency for higher truck concentrations, except for the very large grades.

The entries in Table 3.11 allow the magnitude of T to be calculated and these entries are calculated from:

Equation 3-18

where the denominator corrects for the “additional cars” introduced into the system by the passenger car equivalency factor E_T. The combined effects of restricted lane-width and restricted lateral clearance on capacity are summarized in Table 3.12. The values for W depend on whether an obstruction is on both sides of a highway or just on one side. The entries illustrate that the width correction varies from 1.00 (3.75 m lane widths and no edge restraint) to 0.70 for 2.75 m lane widths and a restraint located at the pavement edge (e.g. a Jersey barrier at a construction site).

The service volume calculated for a given set of conditions is compared to the Design Hourly Volume (DHV) which can be found using Figure 3.18. If the service volume (SV) is found to be larger than the DHV, this indicates that the roadway is sufficient to achieve the level of service goal given condition can be attained. However, if the DHV is larger than the calculated SV than the level of service cannot be achieved in the design traffic conditions.

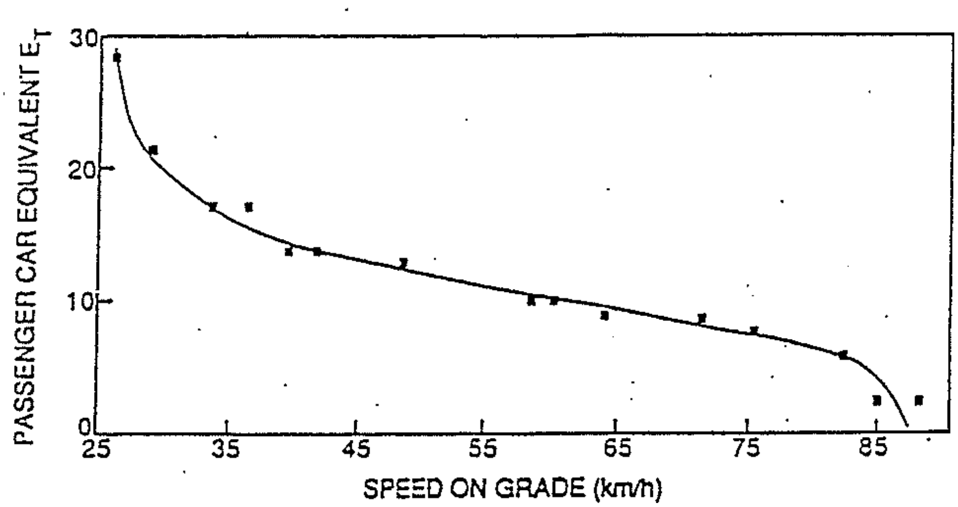


Figure 3.17 Truck Equivalency vs. Speed on Grade

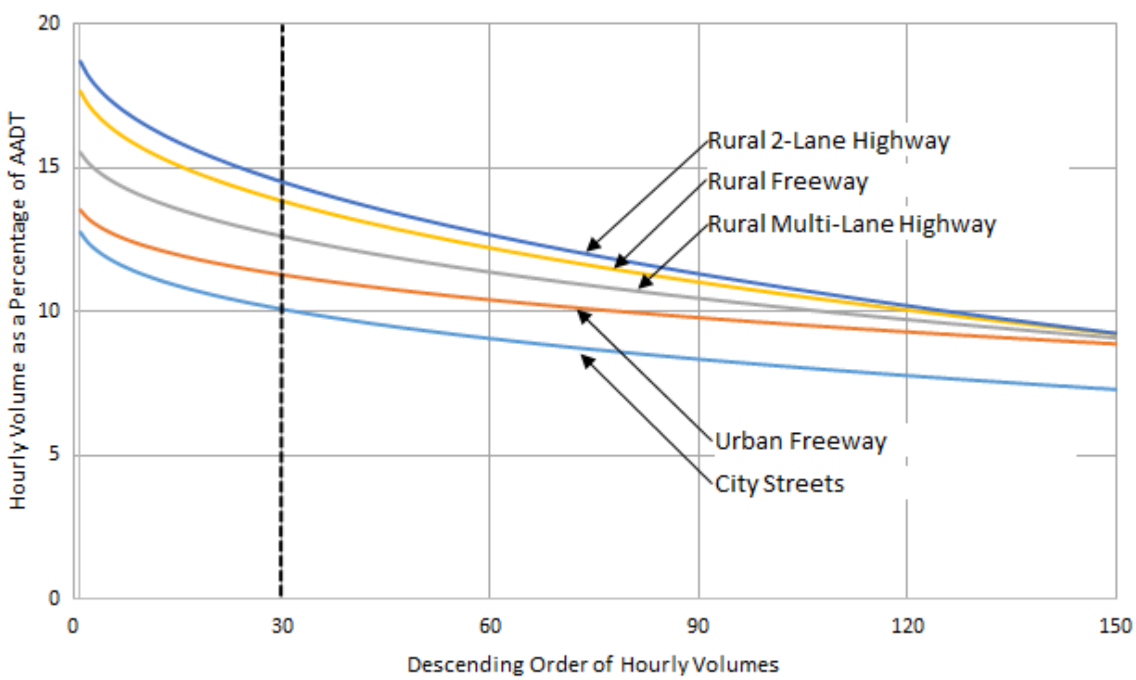


Figure 3.18 Design Hourly Volumes

Example: Number of Lanes Required

Find the number of lanes required at level of service C in the uphill direction of an undivided rural highway at the end of a 1.0 km long, 3% grade. The average highway speed is 100 km/h, lane width is 3.5 m and trucks constitute 7% of the one-way DHV of 2030 v/h. There is a retaining wall located 1.5 m from the edge of the travelled way.

Table 3.9 Levels of Service and Maximum Service Volumes for Multilane Highways

	traffic flow conditions		service volume/capacity (v/c) ratio			maximum service volume under ideal conditions, including 120 km/h AHS (total passenger cars/hr, one direction)		
			basic limiting value[1] of AHS of	value for restricted AHS of				
level of service	description	operating speed[1] km/h	120km/h	100 km/h	80 km/h	(2 lanes one direction	hwy. (3 lanes one direction)	each additonal lane
A	free flow	≥ 100	≤ 0.30	-[2]	-[2]	1200	1800	600
B	stable flow (upper speed range	≥ 90	≤ 0.50	≤ 0.20	-[2]	2000	3000	1000
C	stable flow	≥ 70	≤ 0.80	≤ 0.60	≤ 0.30	3200	4800	1600
D	approaching unstable flow	≥ 55	≤ 0.95	≤ 0.90	≤ 0.70	3800	5700	1900
E[3]	unstable flow	50[4]	≤ 1.0			4000	6000	2000
F	forced flow	< 50[4]		not meaningful		widely variable (0 to capacity)		

[1] operating speed and basic v/c ratio are independent measures of level of service; both limits must be satisfied in any determination of level

[2] operating speed required for this level is not attainable even at low volumes

[3] capacity

[4] approximately

[5] demand volume/capacity ratio may well exceed 1.00, indicating overloading

Table 3.10 Passenger Car Equivalents of Trucks on Multi-Lane Highways on Grades

		Passenger Car equivalent levels of service A through C				
grade, %	length of grade, km	3% trucks	5% trucks	10% trucks	15% trucks	20% trucks
0-1	all	2	2	2	2	2
2	0.5	4	4	4	3	3
	1.0	6	5	5	4	4
	1.5	7	5	5	4	5
	2.0	7	5	5	5	6
	3.0	7	6	6	6	6
	4.0	7	7	7	7	7
	5.0	7	7	8	8	8
	6.0	7	7	8	8	8
3	0.5	10	8	5	4	3
	1.0	10	8	5	5	4
	1.5	10	8	6	5	6
	2.0	10	9	7	6	6
	3.0	10	9	8	7	8
	4.0	10	10	9	9	9
	5.0	10	10	10	10	10
4	0.5	12•	9	5	4	4
	1.0	12•	9	6	6	6
	1.5	12•	10	7	7	8
	2.0	12•	10	9	9	9
	3.0	12•	11	11	11	11
	4.0	12•	11	12	12	12
	5.0	12•	12	13	13	13
	6.0	12•	13	15	15	14
5••	0.5	13•	10	6	5	4
	1.0	13•	11	8	7	8
	1.5	13•	12•	9	9	10
	2.0	13•	13•	11•	11	11
	3.0	13•	14•	14•	14•	13•
	4.0	13•	15•	16•	15•	15•

• values with dot, add 1 for service levels D and E

•• or 6% grade, add 1 (approx.) to E' values for 5% grade

Table 3.11 Adjustment Factors for Trucks on Multi-Lane Highways

passenger car equivalent	Truck adjustment factor T														
	percentage of trucks, P_t or of buses, P_b at:														
E_t or E_b	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26

Table 3.12 Combined Effect of Lane Width and Restricted Lateral Clearance on Capacity and Service Volume of Undivided Multi-Lane Highways with Uninterrupted Flow

distance from traffic lane edge	adjustment factor, [2] *W*, for lateral clearance and lane width									
	obstruction on right side only, of one-direction traveled way (includes allowance for opposing traffic on left)					obstruction on both sides of one direction traveled way[3 4]				
	3.75	3.50	3.25	3.00	2.75	3.75	3.50	3.25	3.00	2.75
	4-lane undivided highway, one direction of travel									
2.0	1.00	0.98	0.94	0.88	0.79	NA	NA	NA	NA	NA
1.5	0.99	0.97	0.92	0.87	0.77	NA	NA	NA	NA	NA
1.0	0.97	0.95	0.91	0.85	0.76	NA	NA	NA	NA	NA
0.5	0.94	0.91	0.88	0.83	0.74	0.93	0.90	0.87	0.83	NA
0.0	0.89	0.87	0.84	0.79	0.70	0.81	0.80	0.77	0.74	0.66
	6-lane undivided highway, one direction of travel									
2.0	1.00	0.98	0.94	0.89	0.79	NA	NA	NA	NA	NA
1.5	1.00	0.97	0.93	0.87	0.77	NA	NA	NA	NA	NA
1.0	0.99	0.96	0.92	0.86	0.75	NA	NA	NA	NA	NA
0.5	0.97	0.94	0.90	0.84	0.74	0.96	0.93	0.88	0.85	NA
0.0	0.95	0.92	0.88	0.82	0.72	0.91	0.89	0.85	0.80	0.70

[1] divided highways, one direction of travel, use adjustment factorfrom Table 4.8

[2] same adjustments for capacity and all levels of service

[3] appropriate for use only where normally undivided roadway is temporarily separated into two roadways by obstructions such as median barriers, bridge structural elements, piers and the like, which are closer thatn would be the opposing traffic.

[4] NA is "not applicable". Use adjustment for obstruction on right side only. In these cases clearance is temporarily greater than the usual separation from opposing traffic but adjustment for this temporary improvement is not feasible.

The capacities of divided multi-lane roads and freeways are analyzed using a similar process. The only differences from that introduced above are the magnitudes of the T and W used for particular combinations of conditions on these road types.

3.11 Two-Lane Highways

Service volumes and capacities for two-lane highways are always calculated for both directions without regard to the distribution of volume by direction. This is because the opposing traffic flows influence the passing opportunities and therefore, capacity. The procedure for the analysis of two-lane highways is expressed in the basic equation:

Equation 3-19

where:

The value v/c depends on average highway speed, level of service and the percentage of available passing opportunities greater than 450 m and the values for v/c ratio are given in Table 3.13. Values of T for various percentages of trucks operating on terrain of different characters are given in Table 3.14. These are generalized adjustment factors for use in the analysis of extended lengths of two-lane highway which may include a number of different grades.

For the analysis of an individual subsections of two-lane highways having a specific grade and length of grade, the passenger car equivalents may be found from Table 3.15

and used to find T from Table 3.16. Values of W for levels of service B and E are obtained directly from Table 3.17. Values for other levels of service are found by interpolation.

3.11.1. Passing Opportunity

For the purposes of capacity analysis, it is assumed that passing opportunities (PO) may occur only where the passing sight distance (SD) is greater than 450 m. This distance is measured from height of driver's eye (1.05 m above pavement surface) to the surface of the pavement.

Availability of PO is expressed as a percentage of the length of road in which the SD is 450 m or more. It is important to remember that the last 450 m from driver's eye to pavement surface, past which the SD becomes less than 450 m should not be included in the length of roadway on which PO is available.

Ordinarily the SD along a highway will vary continually. Where the SD remains 450 m continuously along an extended length of roadway, as across the inside of a long horizontal curve in a cut section, the percentage of availability PO would effectively be less than 100% because some of the drivers would take advantage of passing opportunities only when the continuous SD reaches approximately 600 m. Table 3.18 suggest a rational basis for determining the effective percentage of available PO where the SD is limited continuously to one value.

The value for percentage of available SD greater than 450 m as used in capacity analyses is the average of the percentages for each direction. On sufficiently long sections, however, the available PO will be nearly the same in either direction, and averaging is seldom necessary.

Example: Service Volume

Determine the service volume at level of service C and the capacity of a section of two-lane highway, AHS 80 km/h with 40% passing opportunities (PO) available. The travelled way is 6.0 m with a 0.5 m lateral clearance on each side. The section is in rolling terrain and trucks constitute 20% of the traffic.

Table 3.13 Levels of Service and Maximum Service Volumes for Two-Lane Highways

				services volume/capacity (v/c) ratio					maximum service volume under ideal conditions including 120 km/h AHS. Passenger cars, total, both directions, per hour
	traffic flow conditions		passing opportunity 450 m, %	Basic limiting value for AHS of 120 km/h	working value for restricted average highway speed of				
level of service	description	operating speed km/h			100 km/h	80 km/h	70 km/h	60 km/h	
				≤					
A	free flow	≥ 100	100	0.17	-	-	-	-	340
			80	0.16	-	-	-	-	
			60	0.13	-	-	-	-	
			40	0.12	-	-	-	-	
			20	0.09	-	-	-	-	
			0	0.07	-	-	-	-	
				≤	≤				
B	stable flow (upper speed range)	≥ 80	100	0.47	0.42	-	-	-	920
			80	0.45	0.37	-	-	-	
			60	0.42	0.3	-	-	-	
			40	0.39	0.27	-	-	-	
			20	0.36	0.24	-	-	-	
			0	0.30	0.18	-	-	-	
				≤	≤	≤	≤		
C	stable flow	≥ 65	100	0.73	0.68	0.56	0.48	-	1460
			80	0.71	0.63	0.53	0.43	-	
			60	0.69	0.58	0.47	0.36	-	
			40	0.68	0.54	0.38	0.27	-	
			20	0.66	0.49	0.28	0.27	-	
			0	0.62	0.43	0.18	0.09	-	
				≤	≤	≤	≤	≤	
D	approaching unstable flow	≥ 55	100	0.91	0.88	0.78	0.67	0.57	1820
			80	0.90	0.87	0.75	0.62	0.55	
			60	0.89	0.85	0.74	0.59	0.53	
			40	0.88	0.83	0.72	0.55	0.47	
			20	0.87	0.79	0.69	0.47	0.30	
			0	0.86	0.78	0.67	0.10	0.17	
E	unstable flow	50	not applicable		**≤ 1.00**				2000
F	forced flow	< 50	not applicable		not meaningful				widely variable 0 to capacity

Table 3.14 Average Generalized Adjustment Factors for Trucks on Two-Lane Highways over Extended Section Lengths

	truck adjustment factor, T								
	level terrain			rolling terrain			mountainous terrain		
percentage of trucks, P_t	level of service A	level of service B and C	level of service D and E²	level of service A	level of service B and C	level of service D and E²	level of service A	level of service B and C	level of service D and E²
1	0.98	0.99	0.99	0.97	0.96	0.96	0.94	0.92	0.90
2	0.96	0.97	0.98	0.94	0.93	0.93	0.89	0.85	0.82
3	0.94	0.96	0.97	0.92	0.89	0.89	0.85	0.79	0.75
4	0.93	0.95	0.96	0.89	0.86	0.86	0.81	0.74	0.69
5	0.91	0.93	0.95	0.87	0.83	0.83	0.77	0.69	0.65
6	0.89	0.92	0.94	0.85	0.81	0.81	0.74	0.65	0.60
7	0.88	0.91	0.93	0.83	0.78	0.78	0.70	0.61	0.57
8	0.86	0.90	0.93	0.81	0.76	0.76	0.68	0.58	0.53
9	0.85	0.89	0.92	0.79	0.74	0.74	0.65	0.55	0.50
10	0.83	0.87	0.91	0.77	0.71	0.71	0.63	0.53	0.48
12	0.81	0.85	0.89	0.74	0.68	0.68	0.58	0.48	0.43
14	0.78	0.83	0.88	0.70	0.64	0.64	0.54	0.44	0.39
16	0.76	0.81	0.86	0.68	0.61	0.61	0.51	0.41	0.36
18	0.74	0.80	0.85	0.65	0.58	0.58	0.48	0.38	0.34
20	0.71	0.77	0.83	0.63	0.56	0.56	0.45	0.36	0.31

[1] applicable to buses under most conditions

[2] capacity

Table 3.15 Passenger Car Equivalents of Trucks on Two-Lane Highways on Specific Individual Subsections or Grades

		passenger car equivalent, E_T for all percentages of trucks		
grade, %	**length of grade, km**	**level of service A and B**	**level of service C**	**level of service D and E (capacity)**
0-2	all	2	2	2
3	0.50	7	5	3
	1.00	12	13	11
	1.50	16	20	18
	2.00	18	23	24
	3.00	21	27	28
	4.00	22	29	30
	5.00	22	30	31
	6.00	23	31	32
4	0.5	9	10	7
	1	19	25	26
	1.5	25	34	37
	2	28	37	42
	3	30	41	46
	4	31	43	49
	5	31	44	50
	6	32	45	51
5	0.5	15	19	19
	1	27	39	43
	1.5	32	46	52
	2	35	49	57
	3	37	53	62
	4	38	55	65
	5	39	56	66
	6	40	57	67
6	0.5	28	31	34
	1	36	52	60
	1.5	40	58	69
	2	43	61	73
	3	46	64	79
	4	48	67	83
	5	49	69	85
	6	50	70	86
7	0.5	33	48	56
	1	48	68	80
	1.5	52	73	88
	2	55	77	93
	3	58	81	99
	4	59	83	102
	5	60	85	105
	6	61	87	107

Table 3.16 Adjustment Factors for Trucks on Individual Roadway Subsections or Grades on Two-Lane Highways incorporating Passenger Car Equivalents and Percent Trucks

passenger car equivalent	truck adjustment factor *T*														
	percentage of trucks, P_T														
E_T	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**12**	**14**	**16**	**18**	**20**
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.5
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.92	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.91	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.90	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.89	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.88	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.45	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.77	0.64	0.59	0.54	0.51	0.47	0.44	0.43	0.37	0.34	0.31	0.28	0.26
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.42
26	0.80	0.67	0.57	0.50	0.44	0.40	0.36	0.33	0.31	0.29	0.25	0.22	0.20	0.18	0.17
28	0.79	0.65	0.55	0.48	0.43	0.38	0.35	0.32	0.29	0.27	0.24	0.21	0.19	0.17	0.16
30	0.78	0.63	0.53	0.46	0.41	0.36	0.33	0.30	0.28	0.26	0.22	0.20	0.18	0.16	0.15
35	0.75	0.60	0.49	0.42	0.37	0.33	0.30	0.27	0.25	0.23	0.20	0.17	0.16	0.14	0.13
40	0.72	0.56	0.46	0.39	0.34	0.30	0.27	0.24	0.22	0.20	0.18	0.15	0.14	0.12	0.11
45	0.69	0.53	0.43	0.36	0.31	0.27	0.25	0.22	0.20	0.19	0.16	0.14	0.12	0.11	0.1
50	0.67	0.51	0.40	0.34	0.29	0.25	0.23	0.20	0.18	0.17	0.15	0.13	0.11	0.10	0.09
55	0.65	0.48	0.38	0.32	0.27	0.24	0.21	0.19	0.17	0.16	0.13	0.12	0.10	0.09	0.08
60	0.63	0.46	0.36	0.30	0.25	0.22	0.19	0.17	0.16	0.15	0.12	0.11	0.10	0.09	0.08
65	0.61	0.44	0.34	0.28	0.24	0.21	0.18	0.16	0.15	0.14	0.12	0.10	0.09	0.08	0.07
70	0.59	0.42	0.33	0.27	0.22	0.19	0.17	0.15	0.14	0.13	0.11	0.09	0.08	0.07	0.07
75	0.57	0.40	0.31	0.25	0.21	0.18	0.16	0.14	0.13	0.12	0.10	0.09	0.08	0.07	0.06
80	0.56	0.39	0.30	0.24	0.20	0.17	0.15	0.14	0.12	0.11	0.10	0.08	0.07	0.07	0.06
90	0.53	0.36	0.27	0.22	0.18	0.16	0.14	0.12	0.11	0.10	0.09	0.07	0.07	0.06	0.05
100	0.50	0.34	0.25	0.20	0.17	0.14	0.13	0.11	0.10	0.09	0.08	0.07	0.06	0.06	0.05

computed by $100/(100-P_T + E_R P_R)$

Table 3.17 Combined Effect of Lane Width and Restricted Lateral Clearance on Capacity and Service Volumes of two-lane Highways with Uninterrupted Flow

distance from traffic lane edge to obstruction, m	adjustment factor W_E and W_B for lateral clearance and lane width																			
	obstruction on one side only										obstruction on both sides									
	3.75 m		3.50 m		3.25 m		3.00 m		2.75 m		3.75 m		3.50 m		3.25 m		3.00 m		2.75 m	
	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3	level B	level E^3
2.00	1.00	1.00	0.92	0.94	0.83	0.86	0.77	0.81	0.71	0.77	1.00	1.00	0.94	0.95	0.84	0.87	0.77	0.82	0.72	0.78
1.50	1.00	1.00	0.90	0.91	0.81	0.84	0.74	0.79	0.69	0.75	1.00	1.00	0.88	0.90	0.79	0.83	0.73	0.77	0.68	0.73
1.00	0.98	1.00	0.88	0.87	0.77	0.81	0.71	0.78	0.69	0.72	0.93	0.95	0.80	0.85	0.72	0.78	0.66	0.73	0.61	0.69
0.50	0.95	1.00	0.82	0.84	0.74	0.78	0.68	0.73	0.63	0.69	0.84	0.88	0.72	0.78	0.68	0.72	0.60	0.67	0.56	0.64
0	0.92	0.95	0.78	0.81	0.70	0.75	0.65	0.70	0.60	0.68	0.75	0.81	0.64	0.70	0.58	0.65	0.53	0.61	0.49	0.58

Table 3.18 Effective Passing Opportunity

Continuous SD (m)	Effective Available (PO%)
450	25
500	50
520	70
550	85
580	95
600	100

3.11.2. Climbing Lane Sections

Sections of two-lane highways which include an uphill climbing lane may be analyzed in the same manner as ordinary two-lane sections, with some modification of the procedures for determining the percentage of available PO and the truck adjustment factor T.

Trucks in the uphill climbing lane have no effect on the capacity of the free uphill lane and may be disregarded when determining the truck adjustment factor. Generally, it is sufficient to use one half the actual percentage of trucks in the total traffic volume when selecting the truck adjustment factor in Table 3.16.

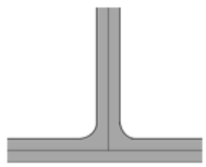
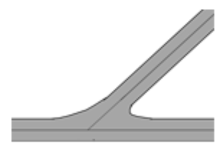
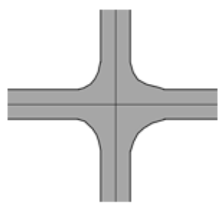
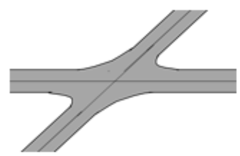
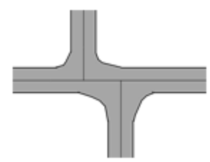
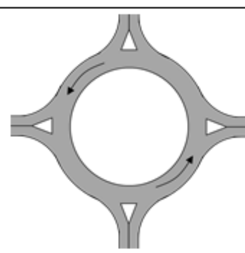
In general, climbing lanes should be provided where the effect of slow moving trucks is to bring the level of service below the desired minimum. Climbing lanes should be extended sufficiently far beyond the top of the grade to permit trucks to accelerate to normal speed before merging with other traffic.

It is important in analyzing an extended length of highway that the length be divided into sections within which the design characteristics are consistent. As a guide, the design speeds on the various segments within each analysis section should not vary more than 20 km/h.

3.12 Intersection Layout

There are four basic types of intersection design and these are shown in Table 3.19. The basic objective of intersection design is to reduce the conflicts between vehicles in order to promote smooth traffic flow and safety.

Table 3.19 Intersection Types

Intersection Type	Illustration
Three Leg	 "T" (Θ = 70° to 110°)  "Y" (Θ < 70°)
Four Leg	 Right Angled  Oblique  Offset
Multi Leg	
Roundabout	

The geometric properties of intersections are governed by the properties of the vehicles using them, such as length, width, and off-tracking. Figure 3.19 illustrates how the length of a vehicle can affect the design of a given intersection. The shaded portion indicates the area of the intersection covered by the truck during the turning manoeuvre. Figure 3.20 shows the off-tracking that can reasonably be expected for trucks based on their wheelbases. "Off-track" refers to the distance between the path followed by the inside wheel on the front steering axle and the path followed by the inside wheel of the last axle. Larger off-track values require truck operators to adjust the paths taken when making a turning maneuver.

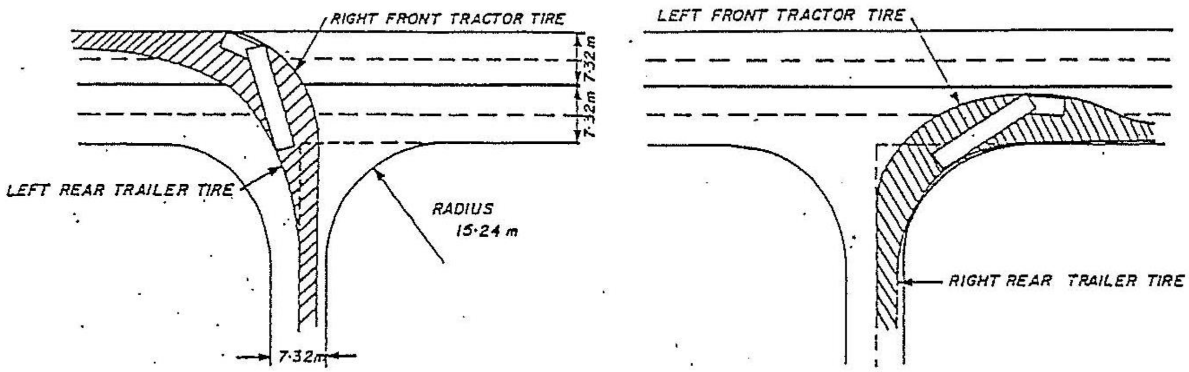


Figure 3.19 Swept Paths of Tractor Semi-Trailers

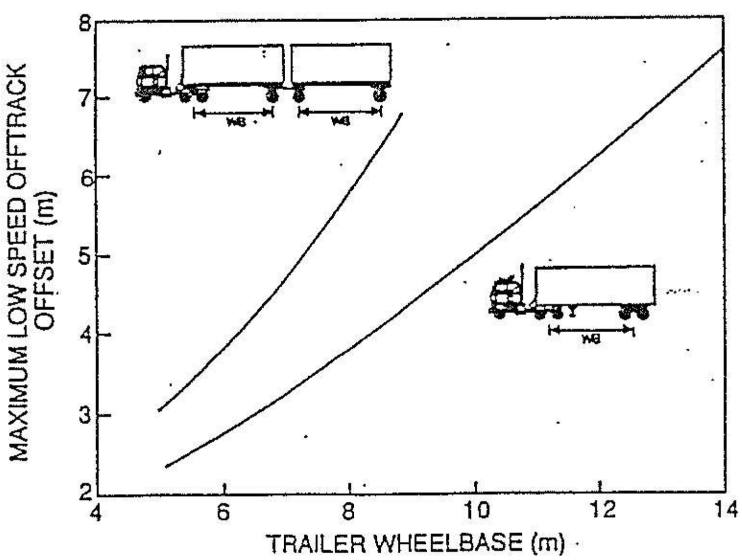


Figure 3.20 Maximum Off-tracking Offset During Low Speed Turns

Five design vehicles are commonly used, and their characteristics are illustrated in Figure 3.21. Changes to vehicle weight and dimension regulations now allow longer trailers with larger off-tracking behaviour. Table 3.20Table 3.20 summarizes the minimum turning radii for various vehicle types. The inner rear wheel does not track on a constant radius and Table 3.20 summarizes the minimum radii at the slowest turning speed.

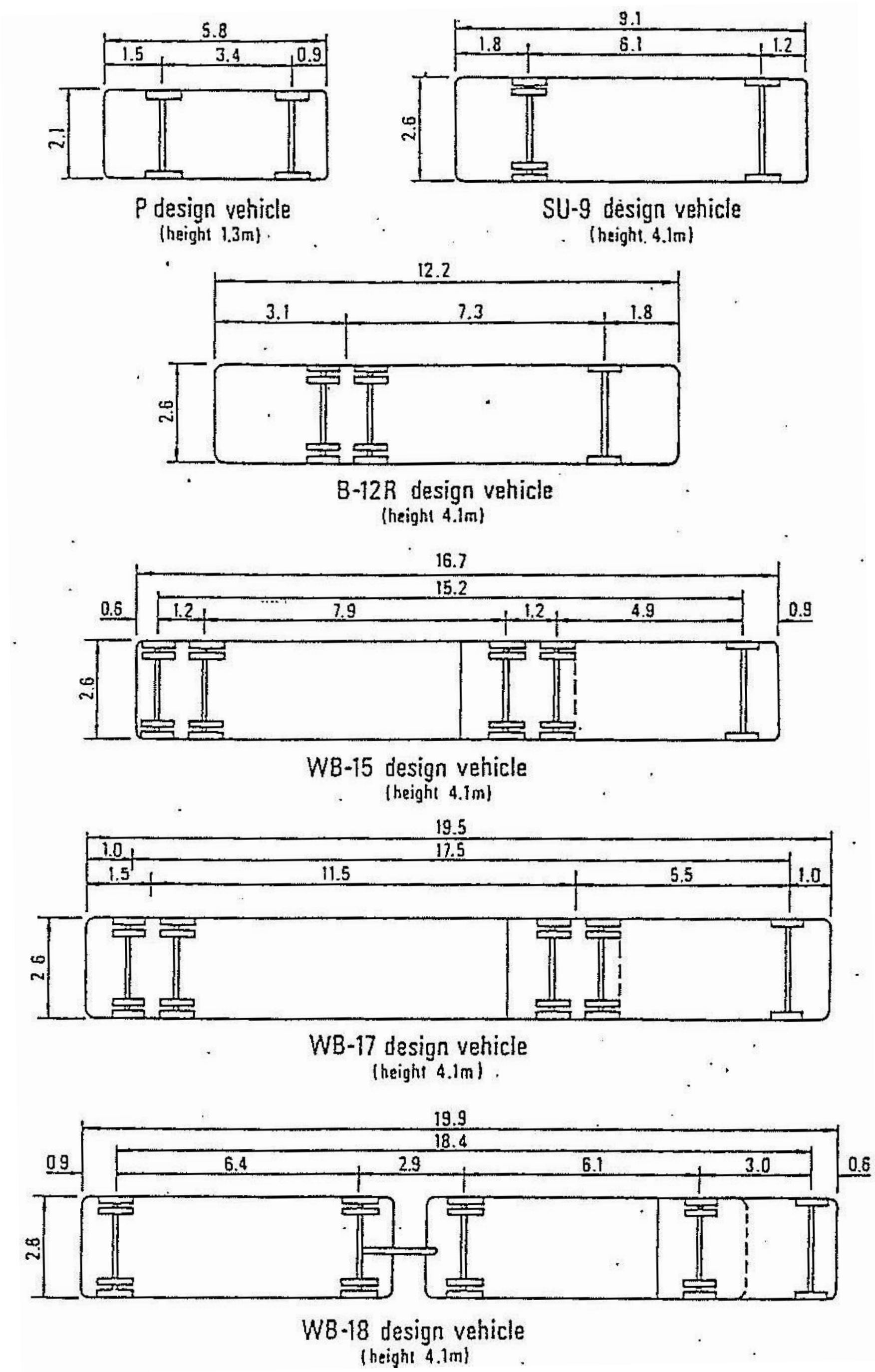


Figure 3.21 Standard Design Vehicles

Table 3.20 Minimum Turning Radii for Design Vehicles

design vehicle type symbol	passenger car P	single unit truck SU-9	highway bus B-12R	semitrailer WB-15	large semitrailer WB-17	semi-full trailer combination WB-18
minimum radii of outer front wheel m						
turning speed 0-15 km/h	7.3	12.8	15.2	13.7	14.6	14.0
turning speed 15-25 km/h	7.3	18.3	19.8	18.3	17.7	17.5
turning speed 25-35 km/h	7.3	18.3	19.8	22.9	22.3	22.2
minimum radii of inner rear wheel	4.7	8.7	7.1	6.0	6.0	6.9
Notes:						
1. Allowance should also be made for the clearance for the outer front overhang which varies from 0.6 m for P vehicles to 0.4 m for WB-18.						
2. Inner rear wheel does not track on a constant radius.						

The minimum turning path and minimum radius curves for four of these design vehicles are shown in Figure 3.22 and Figure 3.23. Normally, turning radii on first class roads are not designed for the minimum geometry because of the impacts on traffic flow. However, off-tracking must be accounted for in all intersection designs. Figure 3.24 shows the typical turning radius for a base design vehicle.

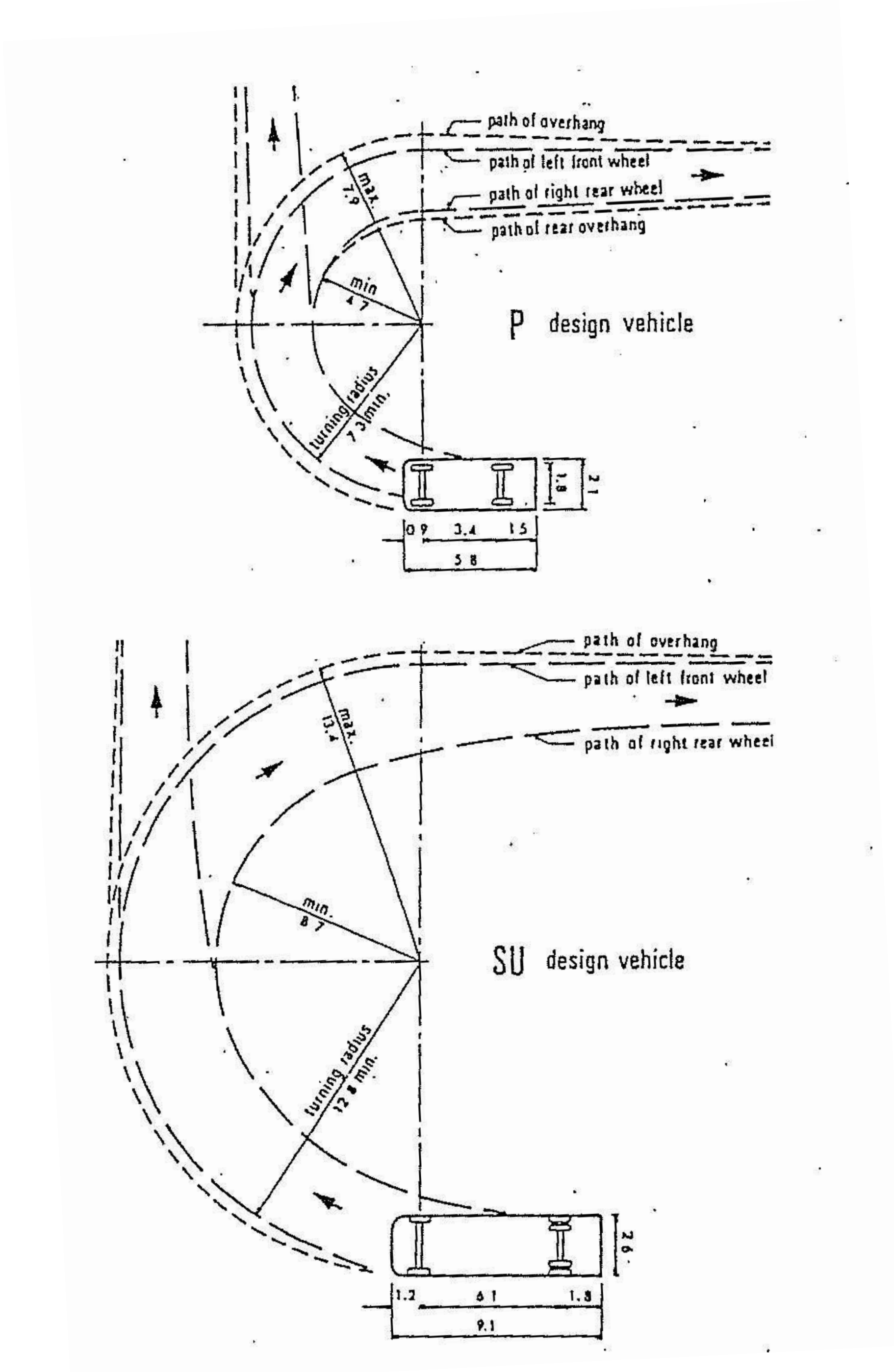


Figure 3.22 Turning Path Characteristics of Design Vehicles

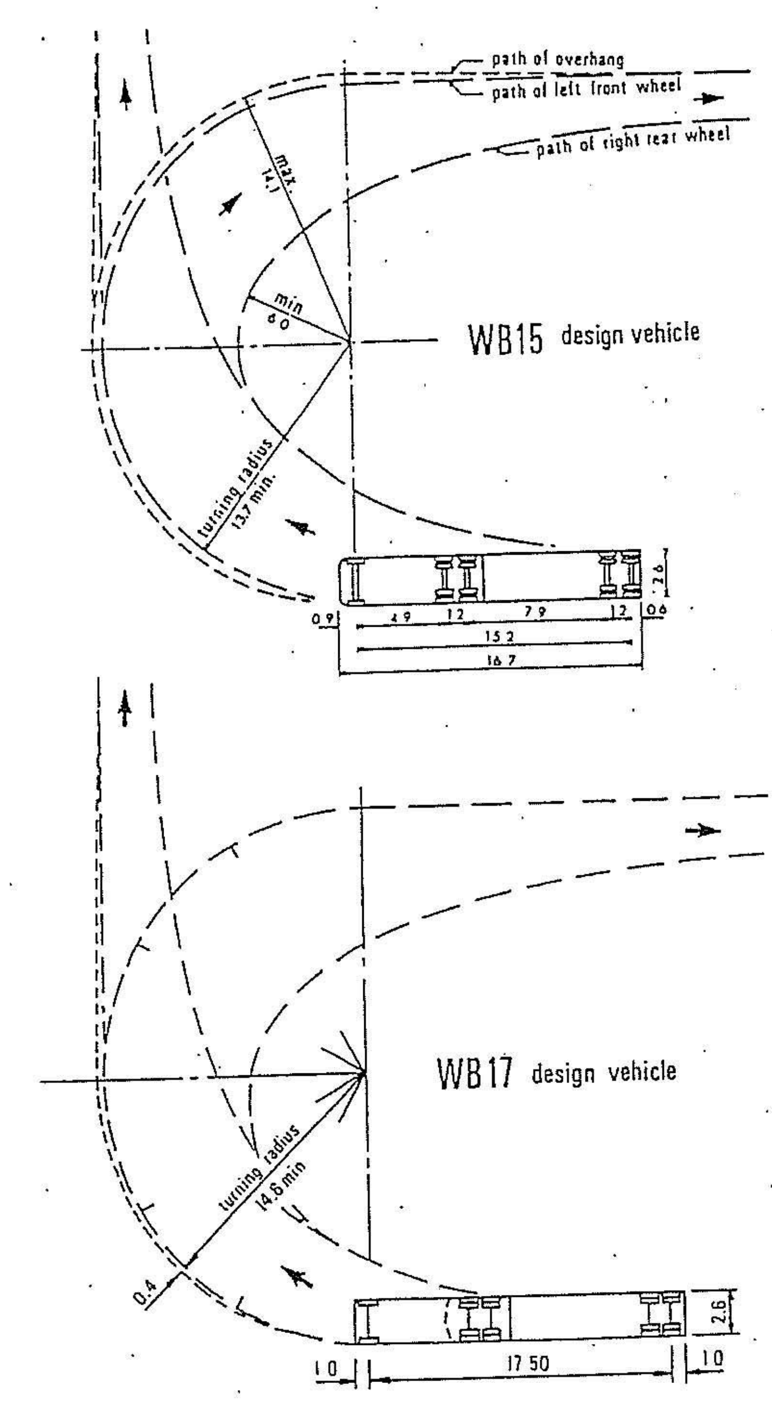


Figure 3.23 Turning Path Characteristics of Design Vehicles

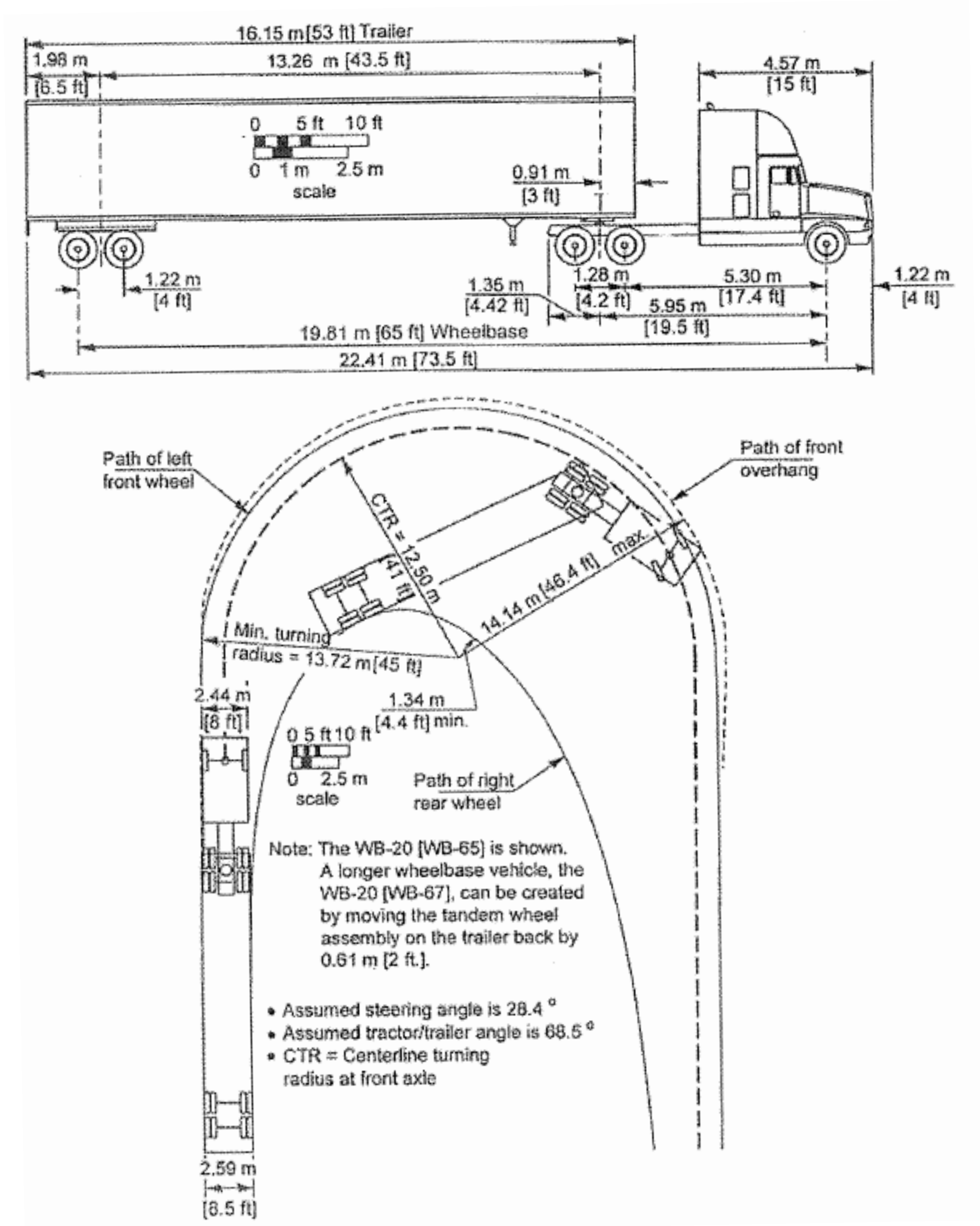
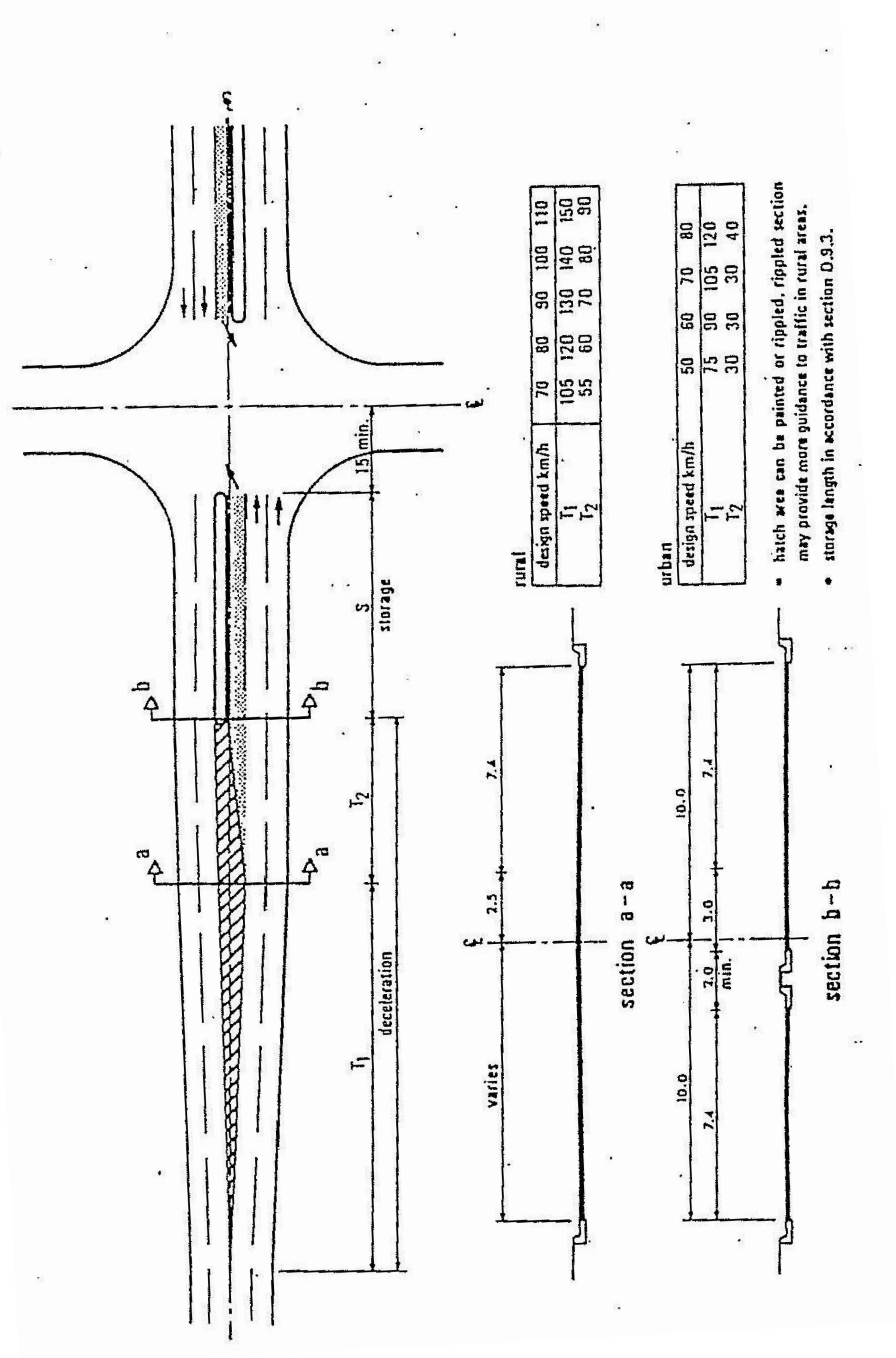


Figure 3.24 Turning Radius for WB-20 Design Vehicle

Figure 3.25 and Figure 3.26 illustrate intersection designs which incorporate storage lanes and these diagrams show the recommended characteristics of these intersections.

Turning lane design, introduced median to minimize hazard.



rural

design speed km/h	70	80	90	100	110
T_1	105	120	130	140	150
T_2	55	60	70	80	90

urban

design speed km/h	50	60	70	80
T_1	75	90	105	120
T_2	30	30	30	40

- hatch area can be painted or rippled, rippled section may provide more guidance to traffic in rural areas.
- storage length in accordance with section D.9.3.

Figure 3.25 Typical Intersection Design

Turning lane design, raised median

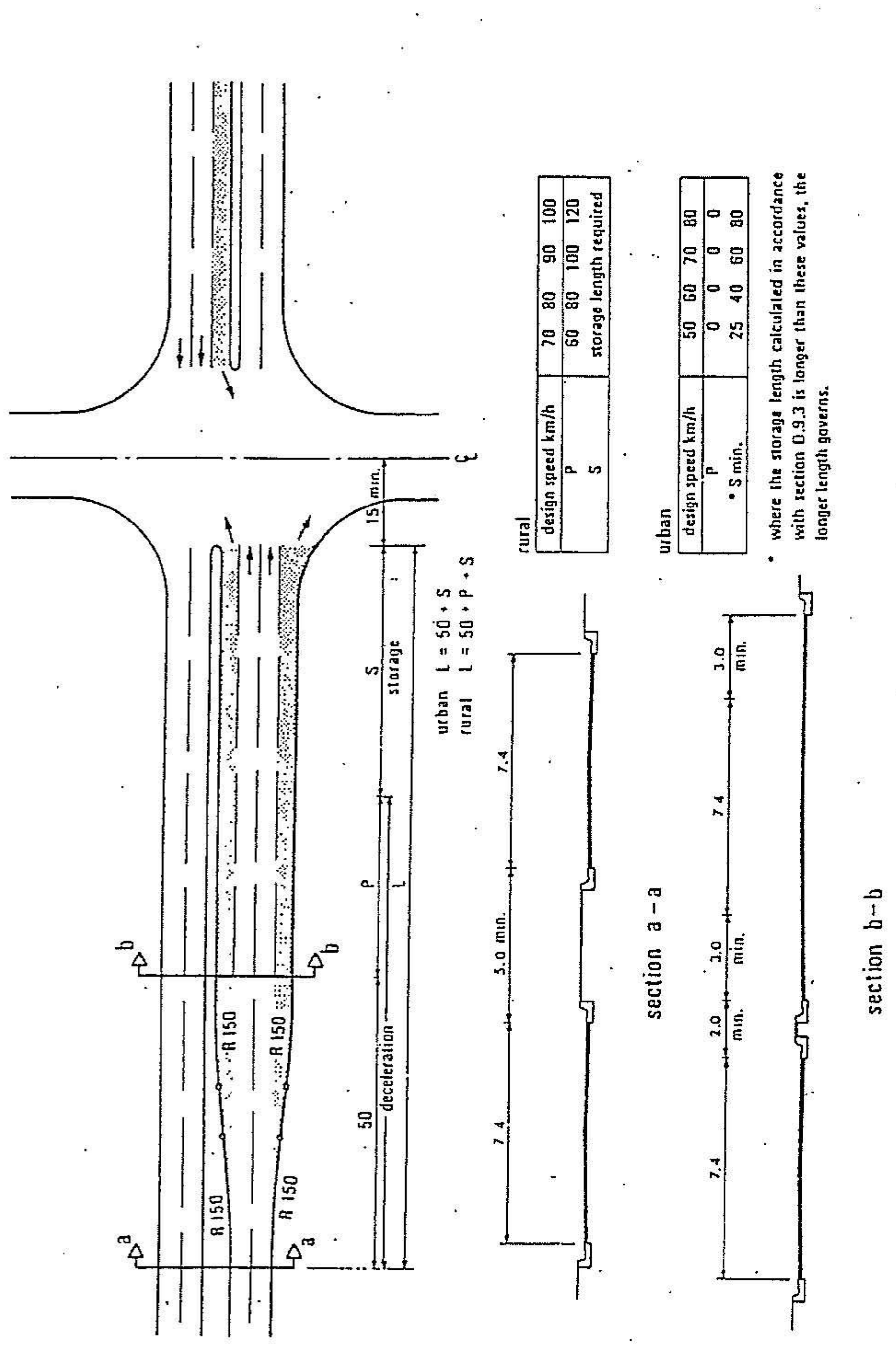


Figure 3.26 Typical Intersection Design with Right Turn Lanes

Roundabout intersections are becoming widely recognized for their safety and flow benefits for medium to heavy traffic volumes. A U.K. program in the 1970's aimed to rehabilitate and upgrade existing roundabouts and many other intersections were converted to roundabouts as well. This served as a watershed moment from which roundabout design, construction, and study was increased substantially.

Studies on roundabouts in comparison to the pre-existing intersections have found significant reductions in the overall number of accidents as well as the proportion of fatal accidents. The following benefits have all been noted about roundabouts (Lenters 2003):

-
-
-
-
-
-

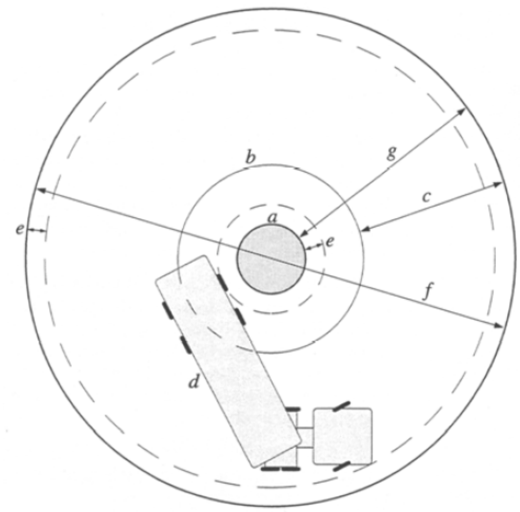
Roundabouts are being more commonly used in Ontario, often to address problems at intersections which have shown significant crash histories in the past. Furthermore, in 2013 the Queen's Highway 406 became the first provincial highway to have a roundabout terminus, which was located in Welland, Ontario.

The Region of Waterloo currently has a by-law which states that roundabouts must be considered when:

-
-
-

The proposed roundabout is analyzed based on safety performance, operational performance, life cycle costs, and qualitative considerations. The life cycle costs (LCC) include the societal costs of injury crashes, which make the safer roundabout design more feasible. The average injury collision cost in Ontario was estimated by Transport Canada to be $82,000.

Furthermore, if the 20-year LCC of the roundabout is within 1.5 times that of a signalized intersection, the roundabout will typically be recommended.



LEGEND

a Raised central island.
b Low profile mountable apron.
c Remaining circulatory roadway width, 1.0-1.2 times the maximum entry width.
d Design vehicle.
e 1 meter clearance minimum.
f Inscribed circle diameter (ICD).
g Width between curbs.

NOTE: Splitter islands should not protrude into the inscribed circle if the roundabout is designed tightly as illustrated here, allowing only the minimum width g.

Inscribed Circle Diameter (f) (metres)	Design Vehicle WB-20 (g) (metres)
79.2	7.2
73.2	7.5
67.1	7.8
61.0	8.1
57.9	8.4
54.9	8.7
51.8	9.0
48.8	9.3
45.7	9.8
42.7	10.1
39.6	11.1
36.6	12.2
33.5	13.7
30.5	**
29.0	**

** Design Vehicle requires larger ICD

Figure 3.27 Roundabout Design Guide for WB-20 Design Vehicle

Example: Roundabout Implementation

Figure 3.28 shows an intersection being considered for the implementation of a roundabout. Due to utilities on the Northwest and southeast corners of the intersection, the available width for the intersection is 44 m. Considering that a minimum of 3.5 m is required outside the pavement edge for curb, bike lane, and ditching, can this intersection support a roundabout that could allow a WB-20 design vehicle through? What other considerations might be relevant?

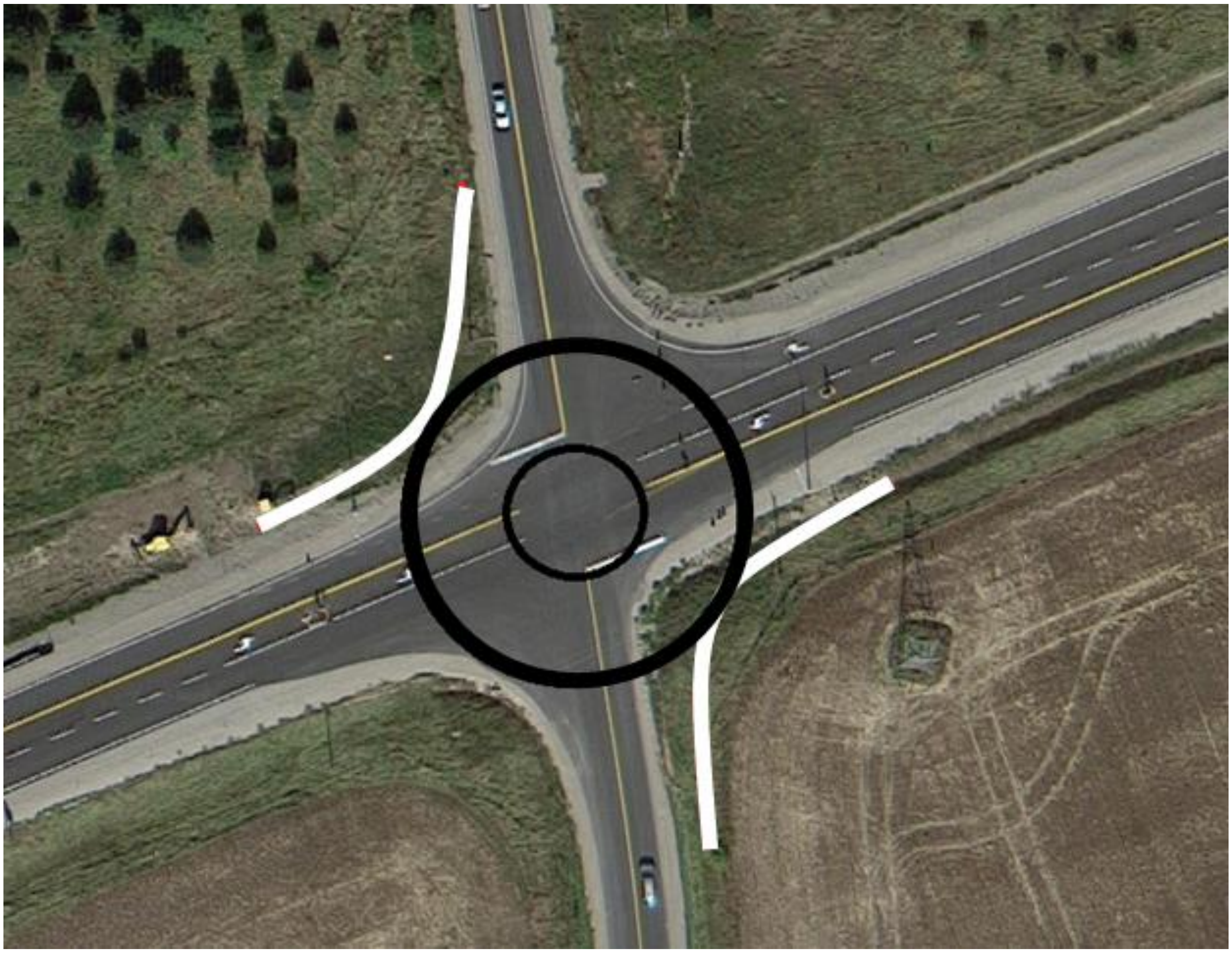


Figure 3.28 Potential Roundabout Intersection Candidate

CHAPTER THREE SUMMARY

Geometric Road Design - Steps to Calculate:

1. Vertical alignment design (SSD, crest curve, sag curve)
2. Horizontal alignment design (consider 2 circular curves with spirals)
3. Capacity analysis (assume level of service D)

1) Calculate the following elements of vertical alignment:
   a) Stopping sight distance
   b) Required length of the crest curve, L_{crest}.
   c) Required length of the sag cure, L_{sag}.
   d) Check to ensure that the two curves fit: Calculate

      Minimum Length = (Lcrest + L sag)/2

      Actual Length = Grade Raise/ % grade

      Minimum Length < Actual Length then it is acceptable

2) Calculate the following elements of horizontal alignment:
   a) Minimum radius of curvature; for e_{max} = 0.06.
   b) Minimum length of the spirals for
      i) comfort
      ii) super-elevation development
      iii) aesthetics requirements

3) Evaluate the capacity of the proposed segment and determine which one is required in order to accommodate the estimated DHV.

3A. Additional Information: Vehicle Properties & Highway Design

ADDENDUM TO CHAPTER THREE

3A.1 Tractive Effort - Tractive Resistance

Figure 3. shows the transmission arrangement of a simple truck. Power is delivered by an internal combustion engine which is translated into torque at the flywheel. The torque is then passed through a transmission gear box with a variety of gear ratios which are designated as “gt” (e.g., 4: 1, which means that four revolutions of the flywheel are required to produce one revolution of the tail shaft). The torque transmitted through the tail shaft then passes through a differential which may have more than one gear ratio available to an operator and the differential gear ratio is usually designated as “gd” (e.g., 5.9: 1, which means that 5.9 revolutions of the tail shaft are required to produce one revolution of the axle. For this combination of gears in the transmission and the differential, 4 x 5.9 = 23.6 revolutions of the engine are required to produce one revolution of the drive axle. The rotating axles then drive the tires and the tractive effort delivered by a truck power unit is transmitted at the tire-pavement interface. The tractive effort that is effectively available depends on the friction that is available between the tire and the road surface.

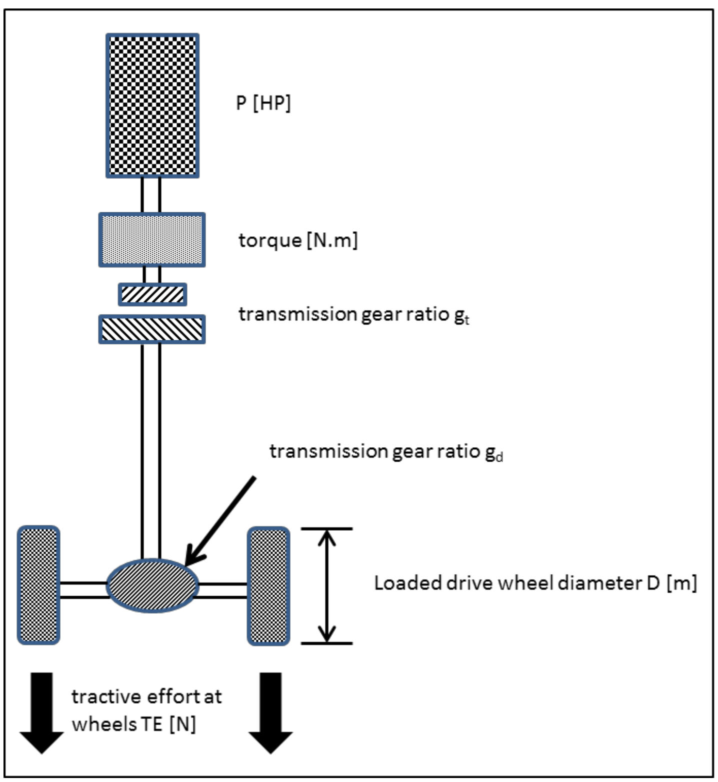


Figure 3.30 Transmission System of Truck

The velocity of a vehicle may be calculated from:

$$V = \frac{60N\pi D}{1000g_t g_d} \qquad \textbf{(Equation 3.20)}$$

where:

- V = the vehicle speed (km/hr)
- N = the engine speed (revolutions/min)
- D = the diameter of the tire (m)
- g_t = transmission gear ratio
- g_d = differential gear ratio

Figure 3.1 shows typical power, torque, and fuel consumption versus engine speed functions for a diesel locomotive engine. The tractive effort developed by a vehicle is given by:

$$TE = 2175\frac{P}{V} \qquad \textbf{(Equation 3.21)}$$

where:

- TE = tractive effort (N)
- P = Power (horsepower)
- V = velocity (km/hr)

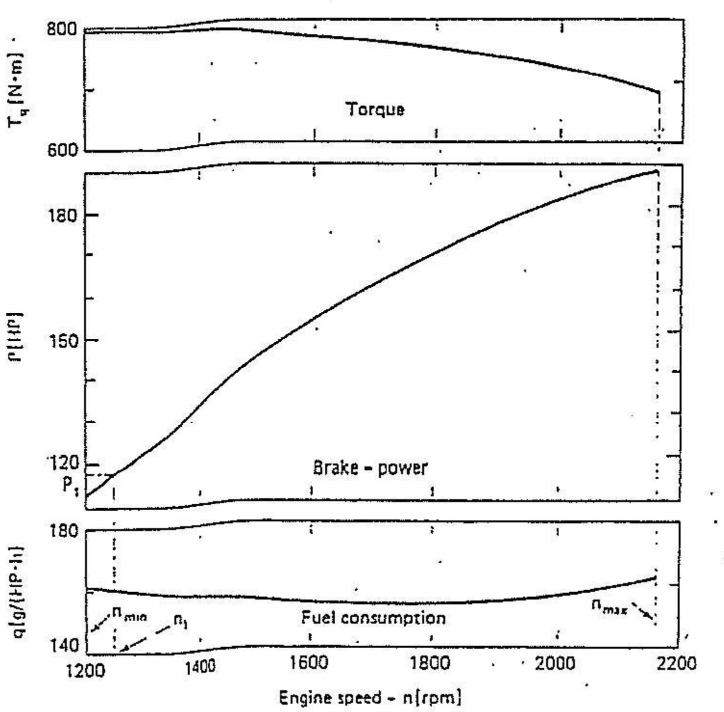


Figure 3.1 Diesel Engine Characteristics

Equations 3.20 and 3.21 may be used to calculate the tractive effort versus speed function for a simple truck by selecting a range of power versus engine speed points from the graph illustrated in Figure 3.1. Three pairs of points are summarized in the first two columns of the upper part of Table 3.1.

The entries in the upper table have, been calculated from eqn. 3.20 using the engine speed information on each line and the varying transmission gear ratios shown at the head of each column. The entries in the lower part of the table have been calculated from eqn. 3.21 using the speeds from the upper part of the table along with the engine powers. That is the entry 29,537 N is calculated from:

$$TE = 2175 \times 110 / 8.1 = 29{,}537 \text{ N}$$

Table 3.1 Calculation of Tractive Effort vs. Vehicle Speed Functions (Entries are km/h)

N (rpm)	P (HP)	Transmission Gear Ratio			
		I	II	III	IV
		6.1	2.9	1.6	1.0
1200	110	8.1	17.1	31.0	49.5
1600	150	10.8	22.8	41.3	66.0
2150	185	14.5	30.6	55.5	88.7

Table 3.22 Tractive effort delivered at different vehicle speeds (V in km/h and TE in N)

Transmission Gear Ratio							
I		II		III		IV	
V	TE	V	TE	V	TE	V	TE
8.1	29,537	17.1	13,991	31.0	7,718	49.5	4,833
10.8	30,208	22.8	14,309	41.3	7,900	66.0	4,943
14.5	27,750	30.6	13,150	55.5	7,250	88.7	4,536

The tractive effort versus speed characteristics of a typical truck are illustrated in Figure 3.2 for three transmission gear ratios. The diagram shows that the tractive effort versus speed function is not continuously variable as it is for the diesel-electric locomotive but provides a discrete range for each gear ratio. The region for each gear ratio shows the complete range of tractive effort versus speed possibilities for various throttle settings, with the upper boundary representing the possibilities for full throttle. Any point below the upper boundary is feasible if the throttle setting is reduced. Points above the upper boundary are not feasible. It is important to note the overlap between adjacent gear ratios.

Example:

Calculate the tractive effort for a vehicle with an engine speed of 1100rev/min, and a horsepower of 110, a tire diameter of 0.8m a g_d ratio of 4:1 and a g_t ratio of 4:1

Given: N = 1100rev/min, P = 110HP, D = 0.8m a g_d ratio of 4:1, g_t ratio of 4:1

Required: Calculate the tractive effort

Analysis:

$V = 60N\pi D/(1000\, g_d\, g_t)$
$= 60(1100)\, \pi(0.8)/[(1000)(4)(4))]$

$= 10.4$

$TE = 2175P/V$
$= 2175(110)/(10.4)$
$= 23{,}005N$
$= 23KN$

$\therefore$ *the tractive effort required would be 23 KN.*

3A.2 Tractive Resistance

The tractive resistance for trucks may be calculated in a similar way to that calculated for trains by the following set of equations:

$$R_r \text{ (rolling resistance)} = (c_1 + c_2 V)\, G/1000 \qquad \text{(Equation 3.3)}$$

where:

R_r = rolling resistance (N)
c_1, c_2 = rolling resistance coefficients (typ. 7.6 and 0.056, respectively)
V = velocity (km/hr)
G = vehicle gross weight (N)

The coefficient c_l reflects the resistance created by tires, bearings, etc., while c_2 reflects the resistance created by the power train components which is a function of velocity.

$$R_a \text{ (drag)} = 0.037\, A\, V^2 \qquad \text{(Equation 3.23)}$$

where:

R_a = drag resistance (N)
A = frontal area (m^2)
V = vehicle velocity (km/hr)

Equation 3.23 is the standard drag equation which is developed in terms of the frontal area A and the vehicle velocity.

$$R_g \text{ (grade)} = 0.01\ G\ m \qquad \text{(Equation 3.24)}$$

where:

R_g = resistance due to grade (N)

G = vehicle gross weight (N)

m = road grade (%)

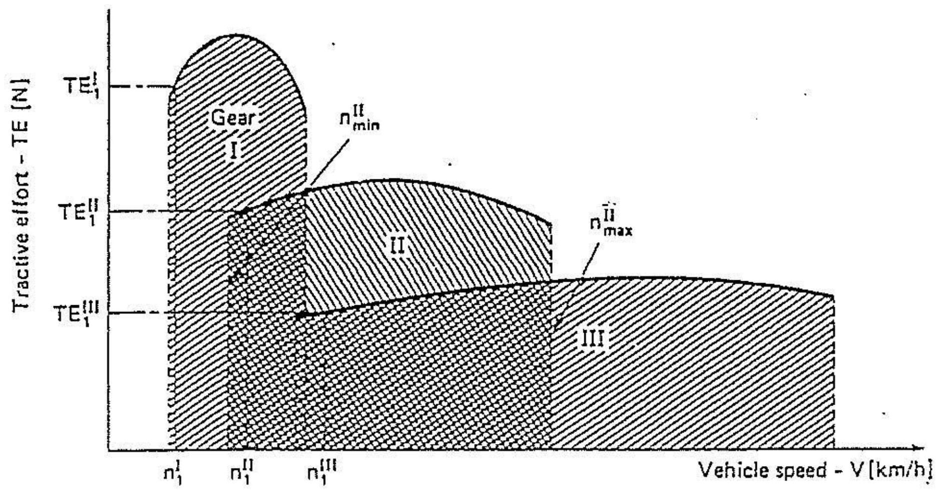


Figure 3.2 Tractive Effort vs. Speed Function

3A.3 Equilibrium Speeds

The equilibrium speeds for trucks may be calculated by equating the tractive effort and tractive resistance. An example equilibrium speed determination is illustrated in Figure 3.3 for a horizontal grade ® and a 2 percent grade (R’). The diagram shows the maximum velocity that can be achieved by this truck in top gear (III). It represents the gear limited speed in that the truck cannot move at a faster speed unless it is on a downgrade. The example shows the reduction in speed that is required on the grade for the tractive effort and tractive resistance to be equal at a speed of V’. The separation of the two tractive resistance functions represents the tractive effort deficit that has to be overcome and the truck will decelerate to a new equilibrium speed. Equation 3.21 shows that a reduction in the speed will increase the tractive effort delivered by the truck.

The transition in speed from v_l to v_2 along a grade may be considered to be with constant deceleration, although this is not strictly true. Figure 3.4 shows some typical speed reduction profiles for trucks on grades of different magnitudes.

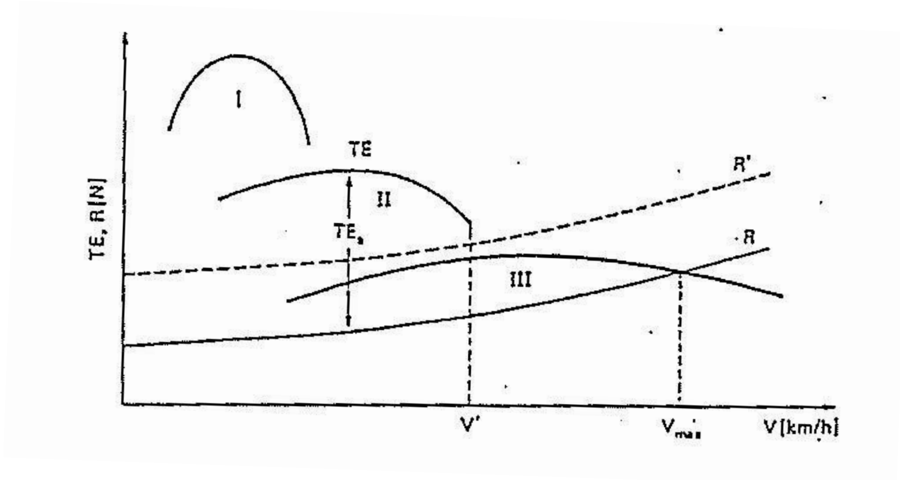


Figure 3.3 Equilibrium Speeds on Different Grades

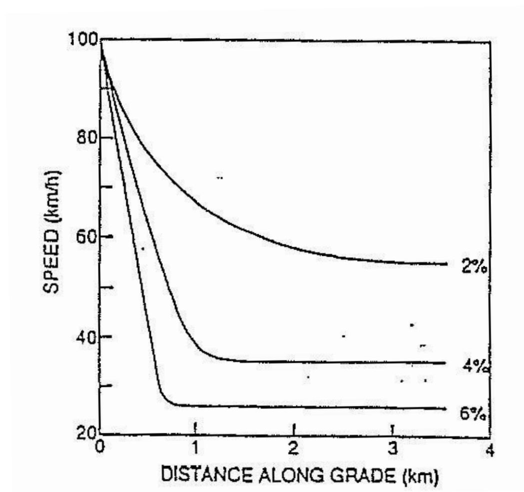


Figure 3.4 Speed Reduction Profiles

These profiles provide insight into why passing lanes become necessary in areas where roadways traverse significant grades. Trucks' velocity decreases over relatively short distances which can cause significant delays if vehicles are "stuck" behind them.

Example:

A tanker truck consisting of 420 HP tractor pulls three trailers. When loaded these trailers have a gross vehicle weight of 25tonnes. The vehicle has the following transmission characteristics (g_t):

gear	1	2	3	4	5	6	7
ratio	15.7	10.3	7.8	4.3.	2.1	1.3	1.0

The horsepower as it relates to the engine speed is described below:

Power (HP)	200	290	350	380	400	420
Revolutions (rpm)	1000	1500	2000	2500	3000	3500

The tire diameter is 1.2m with a differential gear of 5.5:1. Assume the transmission is new. What are the equilibrium speeds at a 2 % and 4% grade? What gears will the vehicle be operating in at these speeds? Comment on the applicability of a 2% and 4% grade for this truck and design speed of 100km/hr.

Given: 100% efficiency of gt

Required: What gears will the vehicle be operating in at these speeds?

Analysis: *Calculate Gross Vehicle Mass (G)* $= 25 * 1000 kg/tonne * 9.8 m/s^2$
$= 245{,}150N$
$= 245.15KN$

Solve for $V = 60N\pi D/(1000\ g_d\ g_t)$ *Equation 3.20 (vary N and* g_t *values)*

Solve for TE = 2175P/V Equation 3.21 (use N and calculated V values with corresponding P values)

Solve for $TR = R_r + R_a + R_g$ *Equation 3.22, 3.23, 3.24*

Determine where TR = TE for a given grade. Summarize the speed and the gear.

3A.4 Braking Characteristics of Trucks

The following is the braking distance formula which assumes that the braking efficiency of the vehicle is perfect:

$$d = \frac{V^2}{254f}$$ **(Equation 3.25)**

where:

d = the distance travelled while braking (m)

V = the velocity at the onset of braking (km/hr)

f = the coefficient of static friction

Perfect braking efficiency means that premature wheel lock-up does not occur at any wheel and this allows the tire-pavement friction to be fully mobilized at each wheel. Figure 3.35 shows the forces acting on a braking wheel. The braking force developed at the tire-pavement interface is denoted by F_B and is given by the product of the normal force and the coefficient of tire-pavement friction f. The braking force is developed by the brake actuating force acting through the brake shoes against the brake drum. The brake force Bf_b is the product of the brake actuating force and the coefficient of friction between the brake lining and the brake drum.

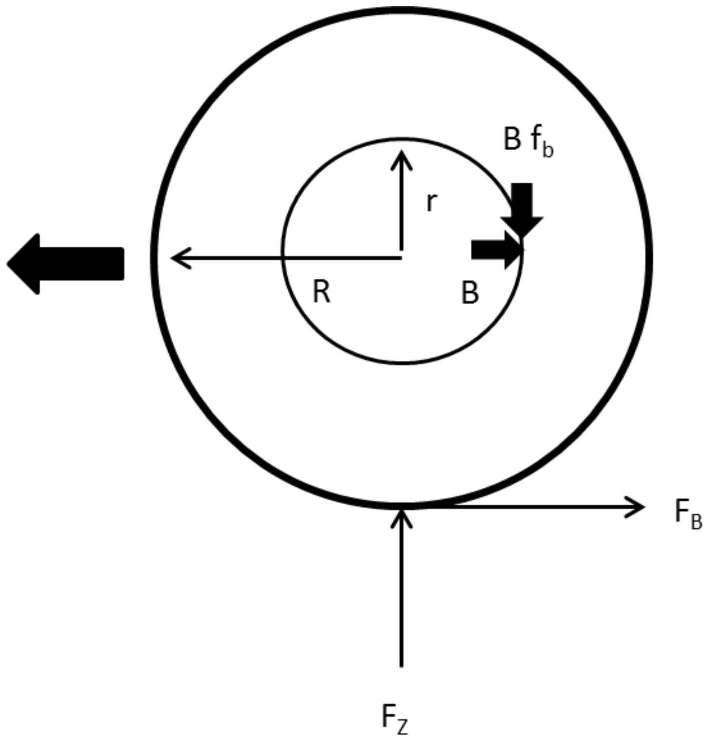


Figure 3.5 Forces Acting on Braking Wheel

Three cases are possible for a braking wheel and these are:

1. the brake torque (Bf_br) = frictional torque (F_zfR) which would utilize fully the available friction at the tire-pavement interface,

2. $Bf_br < F_zfR$, which means that a vehicle is under-braked and the rotation of the tire cannot be prevented, this has been observed on heavily loaded gravel trucks, and

3. $Bf_br > F_zfR$, which means that wheel lock-up will occur and the directional stability of the vehicle cannot be maintained because of the lack of wheel rotation. If this occurs at the tractor drive axle of a tractor semi-trailer then the vehicle would jackknife.

3A.4.1 Ideal Braking

Ideal braking occurs when $Bf_br = F_zfR$ at each wheel. This means that the available tire-pavement friction is being fully utilized and maximum deceleration due to braking is achieved. This condition is impossible to achieve on current trucks under a variety of loading conditions. Brake systems have fixed brake force characteristics and these cannot be adjusted to the different normal loads transmitted through the wheels. Anti-lock systems are available which monitor the rotation of the wheels and release the brakes if rotation stops, although these are not used widely on large trucks.

3A.4.2 Braking of Straight Truck

Figure 3.6 shows the free body diagram for a three-axle straight truck, where the forces acting on the rear tandem are grouped. The braking and normal forces have been defined earlier and the remaining parameters defined in Figure 3.6 are for the dimensions for the wheelbase and the location of the centre of mass.

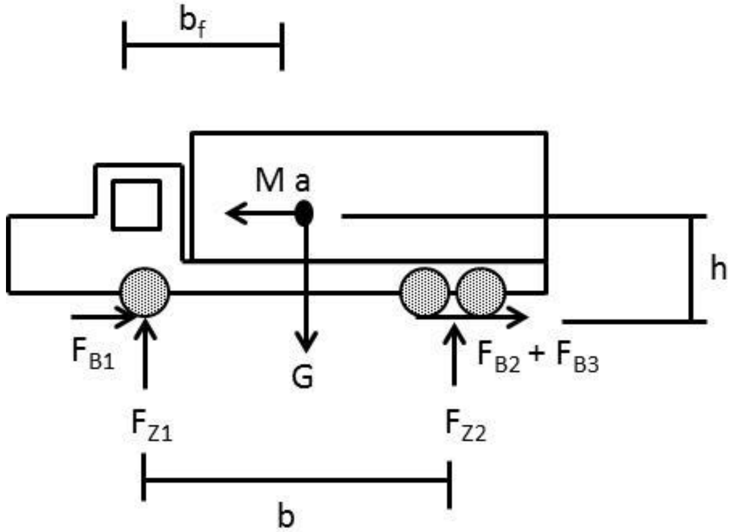


Figure 3.6 Forces acting on Braking Truck

The following equations may be developed from the free body diagram:

$$\boldsymbol{a = \frac{F_{B1}+(F_{B2}+F_{B3})}{G/g}} \quad \text{(Equation 3.26)}$$

$$\boldsymbol{F_{Z2} = \left[Gb_f - \frac{G}{g}ah\right]/b} \quad \text{(Equation 3.27)}$$

$$\boldsymbol{F_{Z1} = G - F_{Z2}} \quad \text{(Equation 3.28)}$$

Equation 3.26 calculates the deceleration of the vehicle by dividing the braking forces developed at the front and rear axle groups by the mass. Equation 3.27 calculates the normal force acting through the rear tandem axle group by taking moments about the front tire point of contact. Finally, Equation 3.28 calculates the normal force at the front axle by subtracting the normal force at the rear tandem from the weight of the vehicle.

This system of equations may be solved incrementally as shown in Table 3.. The vehicle has the following characteristics: G = 268,270 N; M = 27,347 kg; h = 1.50 m; b = 3.83 m; b_f = 2.80m.

The first section of the table shows the braking forces that may be achieved at each axle for various levels of brake pedal depression (these are in increments of 20 psi of brake line pressure). For example, at the first level of pedal pressure the front axle exerts a braking force of 7,565 N and each of the rear axles 13,800 N. The lower braking force at the front axle is designed to ensure that the front axle does not lock-up first and compromise steering control. The increases in braking force at each axle with increasing pedal pressure are illustrated in the upper section of the table. For example, the braking force at each of the rear tandems increases from 13,795 N to 58,740 N.

Table 3.23 Solution of Braking Equations

	Pedal Depression Levels				
Braking Forces (N)	0	1	2	3	4
F_{B1}	0	7,565	16,910	24,030	32,485
F_{B2}	0	13,795	40,050	49,480	58,740
F_{B3}	0	13,795	40,050	49,480	58,740
Normal Forces (N)	**Deceleration (m/s²)**				
	0	1.29	3.55	4.50	5.48
F_{Z1}	72,146	85,962	110,139	120,314	130,879
F_{Z2}	98,062	91,154	79,065	73,978	68,696
F_{Z3}	98,062	91,154	79,065	73,978	68,696
	Required Friction Coefficient to Prevent Wheel Lock-up				
F_{B1}/F_{Z1}	0	0.088	0.154	0.200	0.248
F_{B2}/F_{Z2}	0	0.151	0.507	0.669	0.855
F_{B3}/F_{Z3}	0	0.151	0.507	0.669	0.855

The second section of the table calculates the normal forces acting through each axle at the various deceleration magnitudes. The deceleration magnitudes are calculated from eqn. 3.26 by dividing the sum of the braking forces by the vehicle mass.

Deceleration can be calculated based on known values:

For no braking, the deceleration is 0

Based on the braking forces at Level#1: a= (7565 + 2 x 13795)/268270/9.806 = 1.29 m/s^2

The normal forces at each wheel group are then calculated:

Level #0:
Fz2 + Fz3 = (268270 N x 2.8 m – (268270 N / 9.81m/s^2) x 0 m/s^2 x 1.50m)/3.83m = 196124 N
Fz2 = Fz3 = 98062 N
Fz1 = 268270 N – 2 x 98062 N = 72146 N

For Level #1:
Fz2 + Fz3 = (268270 N x 2.8 m – (268270 N / 9.81m/s^2) x 1.29m/s^2 x 1.50m)/3.83m = 182308 N
Fz2 = Fz3 = 91154 N
Fz1 = 268270 N – 2 x 91154 N = 85962 N

Inspection of the table entries shows that the load increases as the deceleration increases and this results in an unloading of the rear drive tandem. For example, the steering axle load increases from 72,146 N to 130,879 N.

The third section of the table calculates the coefficient of friction magnitude required to prevent wheel lockup. The table entries are obtained by dividing the braking forces at each axle by the normal force.

For Level #1:
FB1/FZ1 = 0.088
FB2/FZ2 = FB3/FZ3 = 0.151

Inspection of the entries shows that the rear axle group is the critical axle group at all levels of brake pedal pressure. At the first pedal pressure a coefficient of friction of 0.151 would be required which is larger than that required at the steering axle. At the highest level of brake pedal pressure a coefficient of friction of 0.855 would be required at the rear tandem axle group to prevent wheel lock-up.

The deceleration that may be achieved by a vehicle under particular tire-pavement conditions may be calculated by interpolation from the above table. For example, if the coefficient of friction is 0.4 then the maximum deceleration would be:

$$1.29 + \frac{0.400 - 0.151}{0.507 - 0.151}(3.55 - 1.29) = 1.76\ m/s^2$$

The braking distance may be calculated from the equations of motion.

The braking efficiency may be calculated from

$$\boldsymbol{n} = \frac{\boldsymbol{a}}{\boldsymbol{g}}\frac{\boldsymbol{1}}{\boldsymbol{f}} \qquad \text{(Equation 3.29)}$$

that is the deceleration achieved in units of g divided by the coefficient of friction. In the above case the braking efficiency is $\frac{1.76}{9.806}\frac{1}{0.4}$ = 0.449 or, 44.9%. Equation 3.29 must be modified to account for the less than perfect braking efficiency of large trucks.

3A.5 Lateral Stability of Horizontal Curves

The following equation for estimating the minimum safe radius for a horizontal curve has been derived earlier in the chapter:

$$R_{min} = \frac{V^2}{127(\mu + e)}$$

The lateral acceleration in a curve is given by (μ + e)g. Using typical values for μ= 0.14 and e = 0.06, the lateral acceleration experienced by a vehicle would be about 0.2 g. This is close to the rollover thresholds of articulated trucks with higher centres of gravity such as tank trucks, where typical values are summarized in Figure 3..

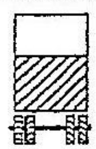
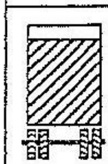
	C of M Height (m)	Rollover Threshold (g)
	2.12	0.34
	2.67	0.24
	2.25	0.32
	2.54	0.26

Figure 3.37 Typical Rollover Threshold Magnitudes

References:

Hoel, L. A., Garber, N. J., & Sadek, A. W. (2008). *Transportation Infrastructure Engineering.* Toronto, ON: Thomson Nelson.

Lenters, M. S. (2003). Roundabout Planning and Design for Efficiency & Safety Case Study: Wilson Street/Meadowbrook Drive/Hamilton Drive City of Hamilton. *Annual Conference of the Transportation Association Of Canada*.

Transportation Association of Canada (TAC). (2013) *Pavement Asset Design and Management Guide*. Ottawa, ON: TAC

4. Pavement Types, Materials and Structure

CHAPTER FOUR

4.1 Introduction

The principal objective of highway pavements is to provide a smooth riding surface with adequate skid resistance over a reasonable period of years, given the load spectrum that a pavement is expected to carry.

This chapter will describe the different types of pavements, importance of material and associated testing, and design methods. In addition, maintenance and rehabilitation procedures and how these relate to life cycle efficiency and environmental considerations will be discussed. The content is primarily based on the 2011 Transportation Association of Canada Pavement Design and Management Guide [TAC 13].

4.2 Types of Pavements

The term “pavement” in Canada generally refers to all the structural elements or layers, including the subbase, base, and asphalt or concrete layers. In other words, anything located above the subgrade or native soils. While the subgrade is not part of the pavement structure per se, its strength and various other characteristics (i.e., drainage, load carrying capacity, etc.) are implicitly included [TAC 11].

Typical types of pavements include rigid, flexible, surface treated, composite, and gravel pavements. The two basic types of pavements are rigid, those constructed of Portland cement concrete (PCC) and flexible, which are constructed with asphalt concrete.

4.3 Pavement Materials

The quality of the materials used in pavement construction, rehabilitation and maintenance, has a significant influence on in-service performance. Those who are involved in pavement design, construction, and maintenance should possess a sound knowledge of construction materials and the related aspects of construction technology and quality control/quality assurance. Pavement engineers and technologists must assure that design, construction, and maintenance operations are undertaken in a manner guaranteed to ensure long term pavement performance. In addition, aspects of drainage that are integral to performance, including surface and sub-surface designs to protect the structure from the negative influences imposed by saturation or inundation, are also considered in this discussion.

Key materials parameters addressed in this Chapter are related to the following major distress mechanisms:

- thermal distress (low and high service temperatures)
- structural distress (fatigue, permanent deformation)
- moisture sensitivity
- compatibility of paving mixture constituents

Material properties or parameters that are described in the following sections are intended to relate to performance, and a number of guidelines are presented. However, regional or local experience, or materials availability, may justify or necessitate deviation from the guidelines. Limits and related provisions specified in this chapter are in accordance with accepted practice. However, individual agencies or pavement engineers should use experience-based judgment in accepting such limitations, or alternatively in modifying any limitation on the basis of local practice or experience.

Nothing contained in this chapter should be considered as a substitute for fundamental knowledge and experience in respect to construction materials technology. Details of project design which relate to geometric requirements are not provided in this chapter, except where they need to be recognized as a function of performance within a regional environment.

The guidelines provided in following sections explain the selection and use of natural and processed materials for construction of the individual components of a completed roadway pavement structure, including subgrade, subbase and base layers and surface layer(s). Materials most commonly used in pavement maintenance and rehabilitation are discussed.

4.4 Subgrades

The subgrade is the underlying or foundation component of the total roadway pavement structure. It is usually constructed with native soil, sometimes in combination with soil imported from local borrow sources. Properties of the as-built subgrade may significantly influence performance of the overlying pavement structure. It is most important that all care and diligence be provided to design and construction of this element of the pavement structure if satisfactory long-term pavement performance is to be ensured. The following discussion addresses factors that are related to selection and use of soils for subgrade or embankment purposes.

4.4.1. Soils Investigations

The Ministry of Transportation of Ontario's Pavement Design and Rehabilitation Manual [MTO 13] describes the purpose of a soil investigation as being to determine the existing geologic conditions and soil engineering properties which will influence roadway design and construction. The specific objectives of soil investigations are to:

- *determine the thickness of topsoil and organic strata.*
- *identify soil types and their engineering properties.*
- *determine the depth to bedrock, if relevant to design and construction.*
- *determine the groundwater conditions.*
- *determine the existing pavement structure layer thicknesses and material properties.*

The Canadian Foundation Engineering Manual published by the Canadian Geotechnical Society [CGS 06] has additional information regarding soils investigations.

Soil tests and sampling should be carried out at spacing intervals that are adequate to delineate the soil stratigraphy reliably and to identify the locations of changes in soil conditions. Therefore, the spacing of test locations and frequencies of sampling should not be fixed or preconceived. They should be a function of in-situ conditions and field personnel should be provided with some level of discretion to position testing locations.

4.4.2. Soil Classification

The Unified Soil Classification System (USCS) is commonly used by geotechnical engineers as a means of characterizing soils according to particle sizes and distribution and plasticities. The tabular form of the USCS is shown in Figure 4.1 and Figure 4.2.

This system was originally developed by A. Casagrande [U.S. Army 53]. It is described in detail in the Canadian Foundation Engineering Manual [CGS 92]. The Unified Soil Classification as shown in Figure 4.1 and Figure 4.2 has been adopted by many highway agencies and is recommended. On a regional basis some agencies have implemented modifications to the original classification system. An example is the characterization of plasticity by denoting silts and clays of low plasticity (CL) as having liquid limit values of less than 30%, and denoting silts and clays of medium plasticity (CI) as having liquid limits values between 30% and 50%.

The system identifies various soil types based on their grain sizes, grain size distributions, and behaviours and properties. The system broadly categorizes the soil types which each include several different subcategories. Generally, the broad categories behave similarly, but different variations will behave differently under certain conditions.

4.4.3. Subgrade Design and Groundwater

It is not the purpose of this chapter to define a preferred method for performing earthworks design. However, a number of special design considerations are frequently necessary when soil properties and/or groundwater conditions exist which have an influence on pavement performance.

4.4.4. Organic Soils

Whenever feasible, organic-rich soils, including peat, should be removed from embankment foundation areas prior to placement of embankment materials. However, it may be necessary, for practical reasons, to “float” or “bridge” embankments over thick deposits of these materials. Geosynthetics, including geotextiles or geo-fabrics and geogrids, are products which have been used in design of such embankments and are discussed later. Other conventional design approaches have been reported for such situations [NCHRP 75].

Major Divisions			Group Symbols (1)	Typical Names	Field Identification Procedures (excluding particles larger than 75 mm and basing fractions on estimated weights)		
1	2		3	4	5		
Coarse grained Soils More than half of material is larger than No. 200 (1) (75 μm) sieve size	Gravels More than half of gravel fraction is larger than No. 4 sieve (4.75 mm)	Clean Gravels (little or no fines)	GW	Well-graded gravels, gravel sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			GP	Poorly graded gravels, gravel-sand mixture, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Gravels with Fines (appreciable amount of fines)	GM	Sifty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
	Sands More than half of coarse fraction is smaller than No. 4 sieve (4.75 mm)	Clean Sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			SP	Poorly graded sands, gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes`missing.		
		Sands with Fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			SC	Clayey sands, sand-clay mixtures.	Plasitic fines (for identification procedures see CL below).		
	(for visual classification, 5 mm may be used as equivalent to the No. 4 sieve size)						
The No. 200 sieve size is about the smallest particle visible with the naked eye.					Identification Procedures on Fraction Smaller tham No. 40 Sieve Size		
					Dry Strength (crushing characteristics)	(Dilatancy (reaction to shaking)	Toughness (consistency near PL)
Fine grained Soils More than half of material is smaller than No. 200 (75 μm) sieve size	Silts and Clays	Liquid limit less than 50	ML	Inorganic silts and very fine sands, rock flour, siity or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight
	Silts and Clays	Liquid limit greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium
			CH	Inorganic clays or high plasticity, fat clays.	High to very high	None	High
			OH	Orgainc clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils			Pt	Peat and other highly organic soils	Readily identified by color, odor, spongy feel, and frequently by fibrous texture		

Figure 4.1 Unified soil classification system Part A

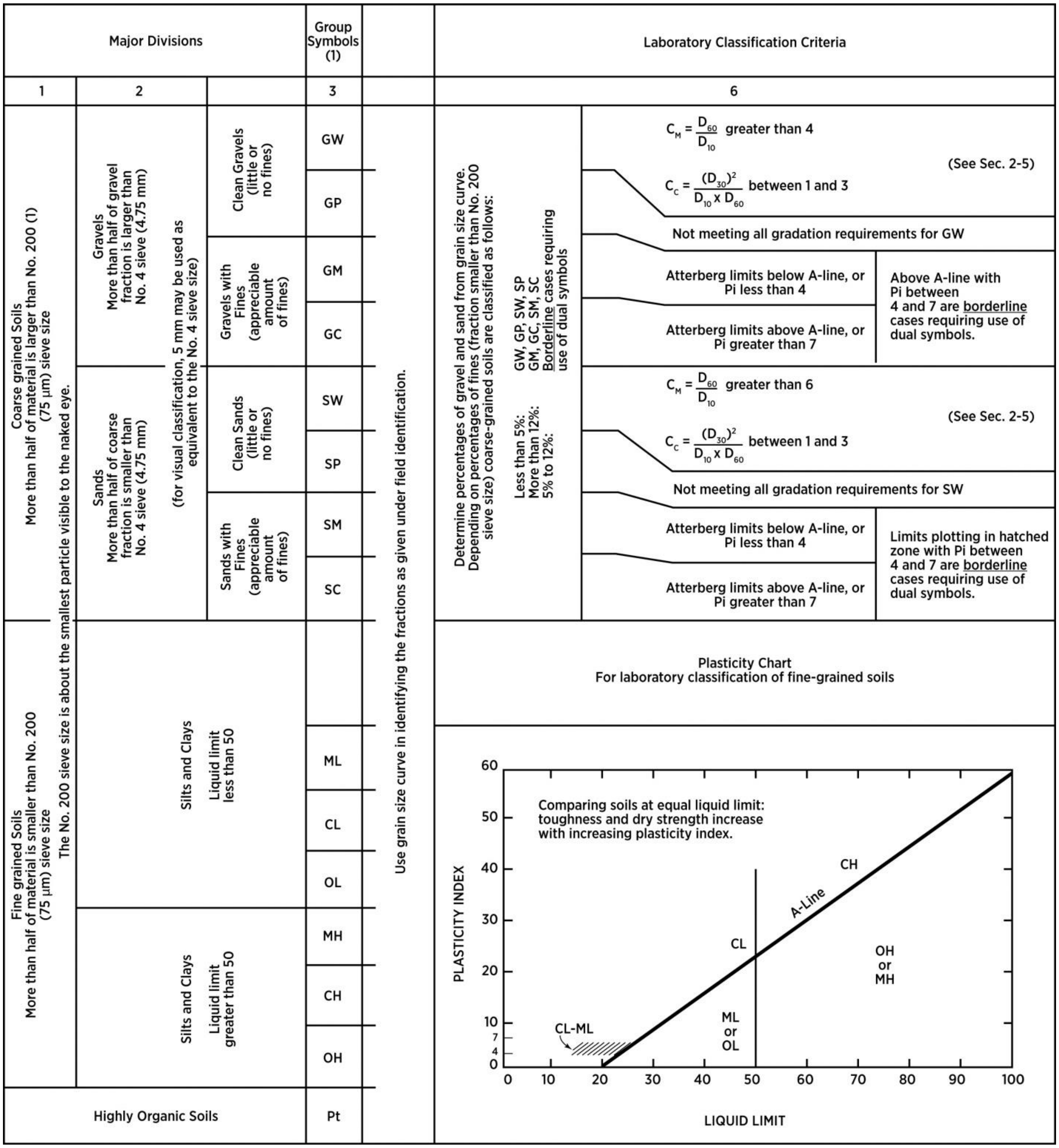


Figure 4.2 Unified soil classification system Part B

Example: Soil Identification

A sample is taken of a subgrade material, and it is found to have 75% passing the No. 200 sieve. The liquid limit is found to be 57, while the dry strength is characterized as high. What soil type could this be? What test might help further clarify this classification?

4.4.5. Frost Susceptible Soils

Frost heave at the pavement surface occurs when ice lenses develop within certain types of subgrade soils as the freezing front penetrates the subgrade, and when a moisture source is available. Mitigation of surface distress due to frost heave necessitates the removal of one of three contributing factors, i.e.,

1. the frost susceptible soil,
2. the freezing condition, and;
3. the moisture source.

Frost heaving can produce pavement surface irregularities due to localized differences in frost susceptibility of soils within an embankment. This type of distress is related to differential frost heave and may be mitigated at the design stage through selective utilization of available embankment materials. The objective is to create embankment uniformity. Competent construction inspection is required during fill placement and compaction, to ensure that uniformity is achieved.

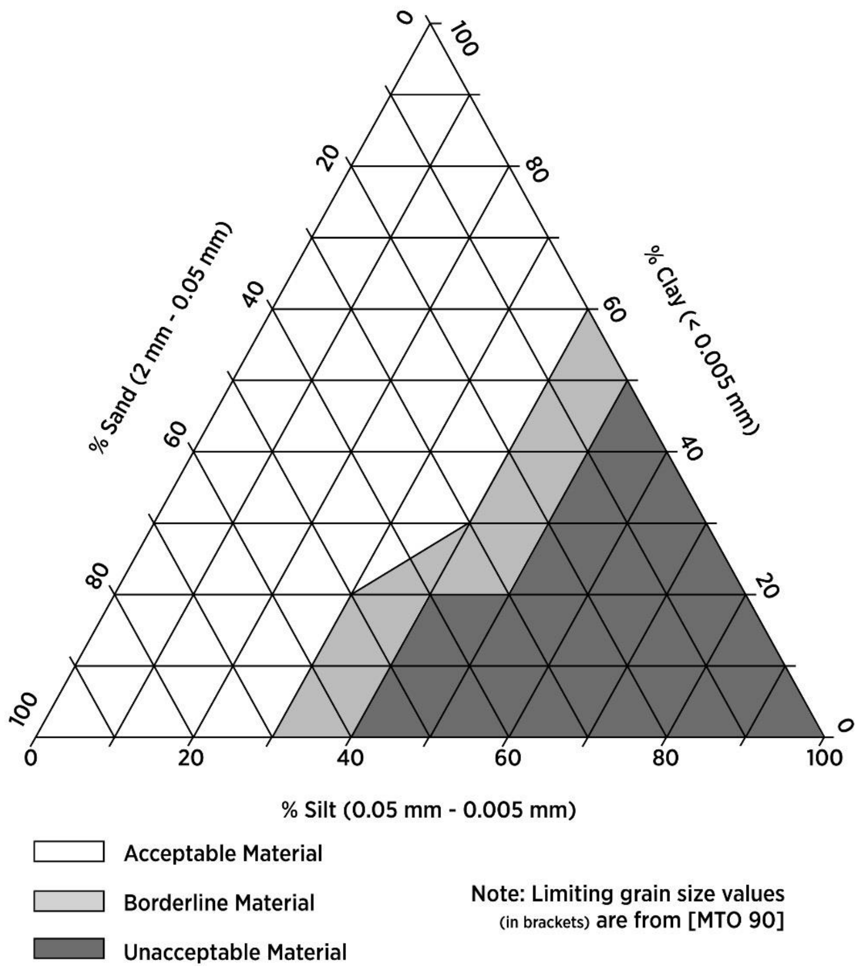


Figure 4.3 Frost susceptibility criteria [Chamberlain 1982]

The Chamberlain method is one of many ways for identifying frost-susceptible soils. The percent composition of the three fine soil types in a sample are located on the nomograph shown in Figure 4.3. If the point is found to be in the unacceptable region, then the soil will typical exhibit frost susceptibility.

Example: Frost Penetration

The fine portion of a soil sample from a job site is found to contain 20% clay, 40% silt, and 40% sand. The design called for this soil type to be within the frost penetration zone. Is this acceptable?

4.4.6. Depth of Frost Penetration

When projects are designed in areas where frost-susceptible soils prevail, and a shallow groundwater condition may exist, it is important to have information on seasonal ground freezing characteristics. The depth of seasonal frost penetration can be monitored using various devices or instrumentation, including frost depth tubes and thermocouple or thermistor strings. Where precision is desired, the latter method is preferable. A data logger can be coupled to a thermistor strong to enable automatic monitoring of ground temperatures at pre-programmed time intervals.

Ontario has developed a design curve of observed frost penetration based on a freezing index [MTO 1990]. The design curve is defined by the following general formula:

Equation 4-1

Where:

P = frost penetration (cm)

F = freezing index °C (degree days Celsius)

The freezing index for specific areas is available from Environment Canada.

4.4.7. Subgrade Height Above Water Level

The desirable height of the top of subgrade (at point of intersection of foreslope) above water level should be set as a function of the subgrade soil type. For normal seasonal high-water level conditions, the height of subgrade may vary from a minimum of about 1.1 m for clean gravel and rock embankments to about 1.8 m (minimum) for embankments constructed with silt and/or clay soils.

Where the maximum recorded high-water level is known, the subgrade embankment height should be at least a minimum of 0.6 m (for clean gravel and rock) and at least 1.2 m (for silt and/or clay) above these levels. Table 4.1 provides the reader with further detail that may be used for establishing design subgrade elevation as a function of water influences.

Table 4.1 Desirable subgrade height above water level

	SOIL TYPE				
	CLEAN GRAVEL & ROCK	SILTY GRAVEL	CLEAN SANDS	SILTY SANDS	CLAYS & SILTS
Subgrade Ht. above design HWL (m)	1.1	1.2	1.1	1.5	1.8
Subgrade Ht. above known HWL (m)	0.6	0.6	0.6	1.0	1.2

4.4.8. Swelling Clays

Certain lacustrine clays contain very active minerals (bentonite and montmorillonite) which give rise to reversible swelling and shrinking properties. These are functions of the soil moisture regime. Use of such soils in subgrades, where seasonal moisture fluctuations occur, can lead to severe distress on the surface of the pavement. Experienced geotechnical engineering expertise should be obtained to confirm the acceptability of suspect soils for embankment construction, and to provide guidelines for design when such soils are encountered.

4.5 Quarried and Granular Aggregates

All pavement structures contain aggregate that is processed either from quarries or from naturally occurring granular sources. The physical properties of processed aggregates are significant to the performance of the total pavement structure. This section contains discussion relative to the geologic origin of processed aggregate materials, methods of assessing the fundamental physical qualities of these materials, and guidelines for their use within pavement structure layers. Processed aggregates are used for constructing the individual components (see Figure 4.4 and Figure 4.5), of the total pavement structure. Typical uses are described as follows.

4.5.1. Subbase Layer

Subbase is the term commonly used to describe the first layer of processed aggregate constructed upon a prepared subgrade or embankment. Its purpose is to transfer traffic-imposed loads from overlying layers to the supporting embankment. It also serves to transmit moisture and to protect the subgrade from seasonal frost effects.

Subbase aggregate may be processed by crushing and/or screening of “pit run” granular aggregate or by quarrying processes. Physical and grain size characteristics which are typically specified would be provided by highway agencies and municipalities.

4.5.2. Base Layer

Base is the term commonly used to describe the layer of processed aggregate constructed upon a prepared subbase (or subgrade). Like the subbase, its purpose is to transfer traffic-imposed loadings from the surface layer(s) to underlying layers of the structure, and to transmit water away from the surface layer(s). Base layers may be either “dense graded” or “open graded”. Aggregates for construction of base layers are normally products of crushing and are of higher quality than those used for the subbase layer.

4.5.3. Flexible Pavement Surface Layer(s)

Aggregate is mixed with products of crude oil refining plus, in some situations, additives or modifiers, to produce asphalt bound pavement layers. The most common type is hot mix asphalt concrete. It is the product of combining aggregate with asphalt cement or enhanced binders (e.g., polymer modified asphalt) in a hot plant mixing process. Common acronyms in much of the industry for this product is HMAC or ACP (for asphalt concrete pavement).

For some low volume traffic applications, aggregate may be mixed with liquid (cutback) asphalt or emulsified asphalt in plants which are designed for such purposes. These mixtures are commonly referred to as "cold mixes", since limited or no heating is required to process them.

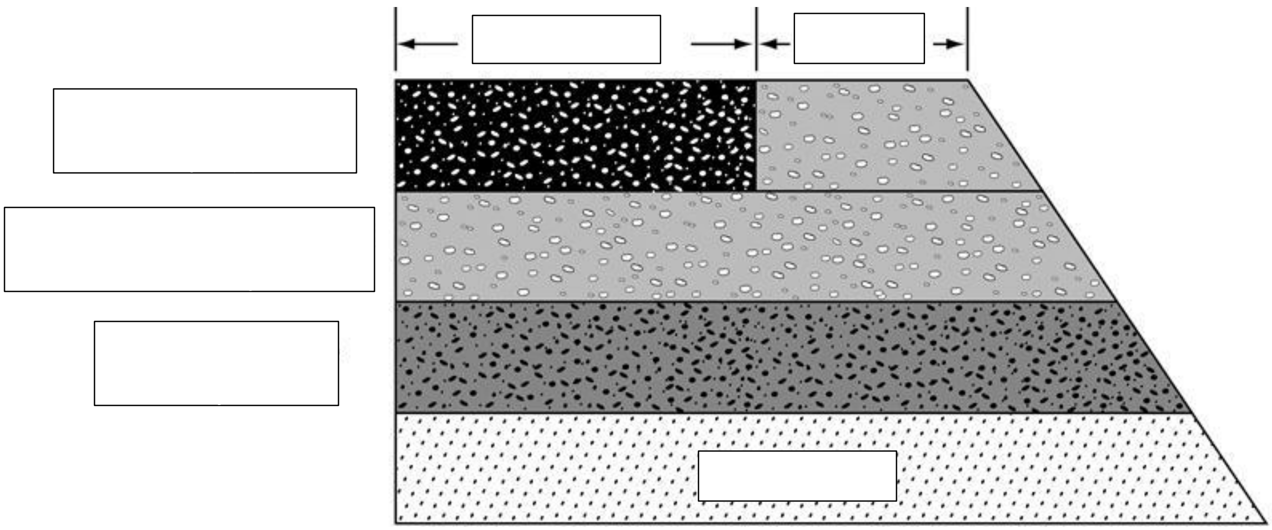


Figure 4.4 Typical cross section of a conventional asphalt concrete pavement

4.5.4. Rigid Pavement

Aggregate is mixed with Portland cement, water, and often other admixtures (including pozzolans and air entraining agents) to produce Portland cement concrete (PCC). Aggregates of best possible quality are usually specified for the manufacture of PCC for road and street purposes.

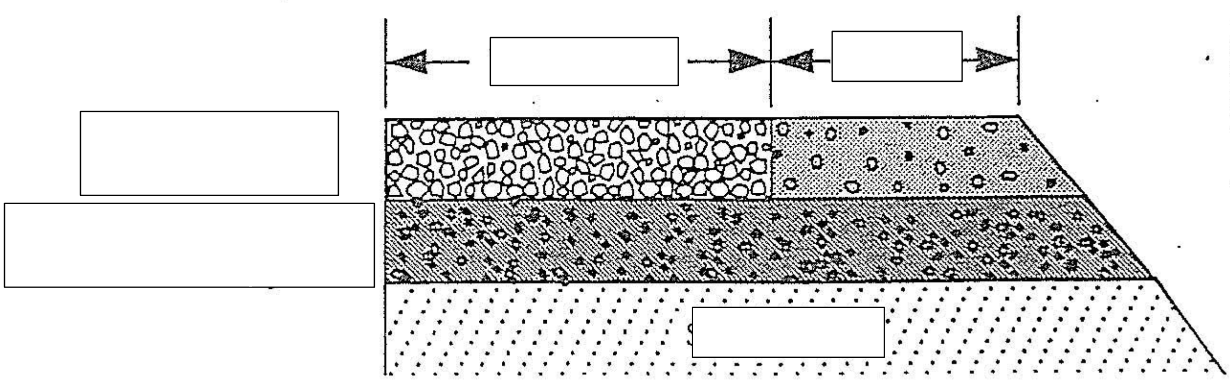


Figure 4.5 Typical cross section of a rigid pavement

4.5.5. Composite Pavement

A pavement structure which combines a relatively thin asphalt concrete layer over a Portland cement concrete layer is produced with aggregates typical of those required for the individual pavement layer types previously described. This type of configuration is commonly referred to as a composite pavement.

4.5.6. Other

Aggregates are used in a variety of processes associated with rehabilitation and maintenance of existing pavements. Commonly used products and processes include cold mix patching mixtures, surface treatments or seal coats, in-place recycling, and other applications.

4.5.7. Physical Properties of Aggregates

The required physical properties of processed aggregates vary depending upon the intended use of either the material itself or products manufactured with the material. Table 4.2 contains typical limiting or specified physical test requirements for base and subbase aggregates.

Table 4.2 and Table 4.3**Error! Reference source not found.** contains typical limiting requirements for aggregates for asphalt concrete mixtures. Tabulated values in Table 4.2 and Table 4.3 encompass the range that may be found in Standard Specifications of typical provincial highway and transportation agencies. A National Standard (CSA A23.1-94) contains similar specification requirements for aggregates for Portland cement concrete.

Table 4.2 Physical properties of base and subbase aggregates

PHYSICAL TEST	ASTM DESIGNATION	TYPICAL LIMITING VALUES FOR		
		GRANULAR BASE	GRANULAR BASE	ASPHALT STABILIZED BASE
Magnesium Sulphate Soundness Loss (% Max)	C88			
Coarse Aggregate		25	20	
Fine Aggregate		30	25	
Los Angeles Abrasion Loss (% Max)	C131	50	45	60
Sand Equivalent (% Min)	D3429	30	40	
Liquid Limit Plasticity Index	D4318 D4318	25 Max. 6 Max.	25 Max. 6 Max.	0
% With At Least One Crushed Face (+ 4.75 mm fraction) (min)	-	-	60	50
Lightweight Particles, % (max)	C123	5	5	
Flat and Elongated Particles (% Max)	4791	15	15	
California Bearing Ratio @ 100% of ASTM D698) (min)	D1883	40	80 (PCC) 100 (ACP)	

Table 4.3 Physical properties of aggregates for asphalt concrete mixtures

PHYSICAL TEST	ASTM DESIGNATION	TYPICAL LIMITING VALUES FOR		
		WEARING COURSE		
		HIGH VOLUME	LOW VOLUME	BASE COURSE
Water absorption, % (max) - Coarse Aggregate - Fine Aggregate	 C127 C128	 2 2	 2 2	 2 2
Plasticity Index, % (max)	D4318	0	0	0
Sand Equivalent, % (min)	D3429	50	45	40
Magnesium Sulphate Soundness Loss, % (max) - Coarse Aggregate - Fine Aggregate	C88	 10 12	 15 18	 18 20
Los Angeles Abrasion Loss, % (max)	C131	25	30	35
Lightweight Particles, % (max)	C123	1.0	1.5	3.0
Loss by Washing Coarse Aggregate (Passing 75 µm sieve), % (max)	C117	1.1	1.0	1.5
Percent Two Crushed Faces (+ 4.75 mm sieve fraction), min.	-	85	70	60
Flat & Elongated Particles % (Max)	D4791	10	10	10

Some of the conventional test procedures have precision limitations, while others have not been proper surrogate tests to represent in-service performance. In recent years a number of tests have been developed which show promise as replacements for existing tests. Test procedures which have been used historically to determine physical properties of aggregates for various purposes are described below. Alternative procedures, which are also now specified, are identified. Two primary reference sources available for theses alternative specifications are the Canadian Standards Association [CSA-A23-14] and the Strategic Highway Research Program [SHRP 94].

4.5.8. Gradation Requirements for Aggregates

It is important to consider that for some pavement applications processed aggregate is the direct construction material (e.g., granular subbase, granular base). However, for other applications, processed aggregate is one of two or more components of the construction material (e.g., Portland cement concrete, asphalt concrete). Therefore, gradation requirements of processed aggregates are variable, depending upon the expected performance.

Aggregate producers must be responsive to a diversity of needs if they are to adequately serve their customers. While it is preferable for a producer to have access to a high quality granular deposit or quarry, it is absolutely essential that the mechanical plant (crushers, screeners, conveying system, wash plant, etc.) is versatile and efficient in order to have the ability to produce graded aggregates to a host of gradation specifications.

At the same time, it is prudent that on a regional or provincial level there is some consistency and uniformity in the specification requirements of various specifying agencies. In Ontario, for example, Provincial Standards for Roads and Municipal Services [OPS – Rev. 92] have been developed. In British Columbia, Master Municipal Specifications [MMS-92] have been the result of a collaborative effort of regional governments, the construction industry and the engineering profession. Such specifications have evolved in consideration of local requirements and the characteristics of regionally available aggregate and quarry sources.

Specification writers should take advantage of existing specifications that are regionally available, whenever possible. Provincial transportation departments, major cities and Canadian government agencies have published standard specifications which are competent to meet most needs. The Canadian Standards Association [CSA-A23-14] provides specifications for Portland cement concrete materials. The Strategic Highway Research Program has published guidelines for products used in new asphalt concrete pavement construction and overlays [SHRP 94].

Table 4.4 Typical gradation requirements for granular base and subbase

SIEVE SIZE (mm) AASHTO M92; ASTM E11	% PASSING				
	SUBBASE	WELL GRADED BASE		OPEN GRADED BASE	ASPHALT STABILIZED BASE
		25 mm	19 mm		
75	100				
37.5				100	
25	55 – 100	100			100
19			100	50 – 100	85 – 100
16		70.90			
12.5				25 – 70	
9.5		50 – 75	50 – 80	15 – 50	50 – 75
4.75	25 – 100	35 – 60	40 – 70	5 – 15	35 – 65
2.36	15 – 80				
1.18	15 – 45	15 – 40	15 – 40	0 – 8	15 – 40
0.600	10 – 35				
0.315		8 – 20	8 – 25		5 – 25
0.150	5 – 15	5 – 15	5 – 18		
0.075	0 – 8	2 – 8[1]	2 – 8[1]	0 – 5	2 – 8

1 *The upper limit on this size is reduced to 5% by some provincial agencies in order to improve layer permeability properties*

Aggregate gradation requirements specified by Canadian transportation agencies have similarity in respect to the most commonly used products in construction of pavements. There is not, however, a uniform set of standards, or indeed products, that prevails throughout the country. This is reasonable to expect since aggregate sources are not uniform, and regional experience has accommodate use of the available aggregate resources. Table 4.4 contains typical gradation requirements for granular base and subbase materials most commonly specified. Table 4.5 contains typical gradation requirements for granular aggregates for asphalt concrete paving mixtures.

Table 4.5 Gradation requirements granular aggregates for asphalt concrete paving mixtures

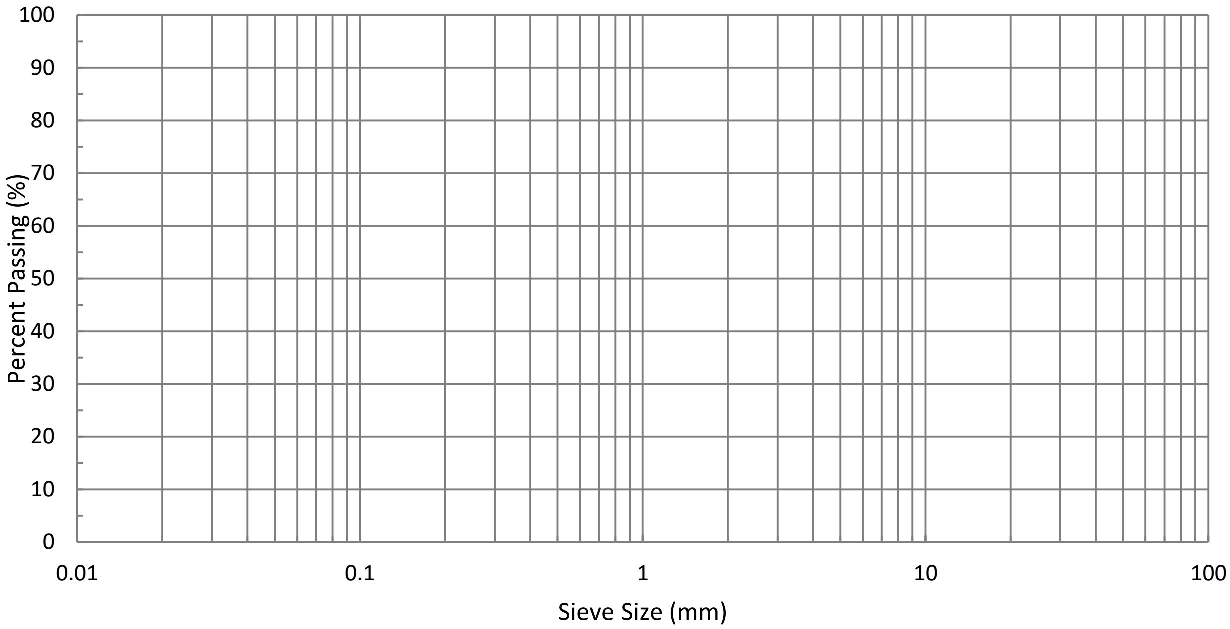
Sieve Size (mm) AASHTO M92; ASTM E11	Base Mix		Asphalt Bound Open Graded Base Mix	Dense Surface Mix	
	25 mm	19 mm		19 mm	12.5 mm
31.5	100				
25	90 – 100	100	100	100	
19	70 – 95	90 – 100	75 – 100	90 – 100	100
16					
12.5	60 – 85	65 – 85			90 – 100
9.5	50 – 78		30 – 60	56 – 80	
4.75	30 – 60	35 – 65	5 – 30	35 – 65	45 – 75
2.36	20 – 50	25 – 55	0 – 10	23 – 49	28 – 58
1.18	15 – 40	15 – 45			
0.600	10 – 30	10 – 35			
0.300	5 – 20	5 – 25	0 – 8	5 – 20	5 – 21
0.150	1 – 15	1 – 15			
0.075	0 – 8	0 – 6	0 – 5	2 – 8	2 - 8

Example: Aggregate Gradation

SIEVE SIZE (mm) AASHTO M92; ASTM E11	**75**	**37.5**	**25**	**19**	**16**	**12.5**	**9.5**	**4.75**	**2.36**	**1.18**	**0.600**	**0.315**	**0.150**	**0.074**
Percent Passing	**100**	**100**	**100**	**94**	**92**	**71**	**65**	**40**	**20**	**18**	**14**	**8**	**4**	**3**

The gradation for an aggregate source is shown in the table above. The aggregate supplier would like to use this material to produce a 19 mm base mix. Does the material currently meet the gradation requirements? If not, how could the material be altered to meet the specifications? Fill in the chart.

2.36 mm will be outside of boundary 20 vs 25-55. Therefore, the material is too coarse slightly. Removing the 19 mm fraction would shift everything up by 6%, could be something to consider. Generally, the amount of extra preparation is tried to be minimized at the producer. The engineer could use his judgement to determine whether this aggregate source is acceptable or not.



4.6 Bituminous Materials

Bituminous materials used for paving and related purposes are products of the refining of crude petroleum or naturally occurring asphalts. Asphalt, a residue product in the refining of crude oil, is a dark brown to black solid or semi-solid cementitious material that gradually liquefies when heated and is usually called petroleum asphalt. Asphalt cement is a product that has been refined further to meet specifications for paving and related purposes. It is the primary product used in the construction of paved roads, and is the binder used in all asphalt concrete materials.

Liquid asphalt includes cut-back and emulsified asphalts, both of which are liquid at or near ambient temperature. Cut back asphalt is asphalt cement which has been liquefied by blending with petroleum solvents or oils of low to high volatility. Emulsified asphalt is a mixture of asphalt cement, with or without petroleum solvent, and water containing an emulsifying agent, which maintains the asphalt globules in suspension. Cut back asphalt is declining because of environmental concerns.

Asphalt cement consists of asphaltenes, maltenes, asphaltic resins and oily constituents. The characteristics of asphalt cement depend largely upon the relative proportions of these three constituents, which can vary based on the crude oil source. It is important to note that asphaltenes are used in research and evaluated when examining the refining of special additives. Asphalt specifications were developed to define physical properties such as; penetration, viscosity and ductility. A criticism of these conventional physical property tests was that they were performed at standard temperatures, regardless of the environmental characteristics of the intended area of use.

As an example, the penetration test is performed at 25°C, while viscosity testing is performed at temperatures of 60° and 135°C. Depending on the geographical location, the range of temperatures may not reflect the thermal conditions in which the binder is expected to perform.

In Canada, asphalt cements have historically been graded in accordance with their hardness, as defined by a standard penetration test [ASTM 2006e]. The higher the penetration number, the softer the asphalt cement. An important consideration of the penetration grade system is that the specified requirements are for original (as-supplied) products and do not incorporate either initial age hardening that occurs in hot mix production, or in long-term, in-service aging. In Canada, a pen-vis type specification has been used successfully in evaluating various asphalt cements, it is notable that this specification is more robust than the ASTM and asphalt penetration/viscosity specifications. The pen-vis specifications from either the CGSB or individual provinces incorporates initial age hardening through the use of a thin film oven test and retained viscosity and mass loss specifications. However, it does not include long-term aging.

For use in the production of asphalt concrete paving mixtures, the penetration grade to be used on a specific project is dependent upon a number of factors including:

1 Seasonal low temperature regime which will influence the development of low temperature transverse cracking in the pavement structure.

2 Seasonal high temperature regime which will influence the development of permanent deformation of the pavement under heavy truck traffic.

3 Traffic (type and volume).

In selecting the grade of asphalt cement for a specific project, sufficient knowledge should exist of the technology associated with the determination of asphalt binder and paving mixture stiffness values under specific temperature and loading conditions. For larger provincial and local agencies this knowledge or expertise will likely exist in-house [TAC 13].

4.6.1. Impacts of SHRP Research

To address the shortcomings of the traditional conventional asphalt grading subsystems, such as ASTM and AASHTO, the Strategic Highway Research Program (SHRP) in the United States and the Canadian counterpart (C-SHRP) undertook, as a primary objective, development of performance-based grading specifications for asphalt binders. Testing protocols were developed to best represent actual field service conditions in asphalt pavements. [TAC 97]

4.6.2. Superpave Asphalt Binder Specification

The result of this research was the development of a new asphalt mixture design and analysis system called Superior Performing Asphalt Pavement (SuperPave) that includes a performance-based asphalt binder specification and a mix design analysis system.

The resultant performance grade asphalt cement (PGAC) specification, classifies asphalt binders based on the temperature environment for their intended use. Asphalt binders are performance graded and are required to comply with specified requirements at both the low and high pavement in-service temperatures which must be determined on a project specific basis. For example, a binder identified as PG 58-34 must meet performance criteria at an average seven-day maximum pavement design temperature of 58°C, and also at a minimum one day pavement temperature of -34°C.

4.7 Portland Cement

Portland cement clinker is made by grinding together selected proportions of an argillaceous material (consisting largely of oxides of silica, iron and aluminum) with a calcareous material (such as limestone) that supplies calcium oxide. The mixture is passed through a rotary kiln at temperatures up to 1600^{o}C. Portland cement is an interground mixture of cement clinker and a predetermined amount of gypsum that is required for controlling setting time and obtaining optimum strength. The term “Portland” or “Portland” cement originated as a trade name and thus it gives no indication of composition or properties [Mindess 81]. The term now applies to a number of closely related cements that have generally similar properties.

Portland cement is composed primarily of calcium compounds (silicates, aluminate, ferric oxide, gypsum). When Portland cement is mixed with water its constituent compounds go through chemical reactions which cause eventual hardening of the concrete. These reactions are termed hydration. It is the rate of hydration and products of the hydration process which yield the properties required by the designer or specifier. The hydration process, and resultant strength gain, continues long after initial hydration, although usually about two-thirds of the ultimate strength is attained in about seven days after placement, if the concrete is properly protected.

Different Portland cements can be made for particular applications by modifying certain properties. Portland cement is specified in Canada by the Canadian Standards Association (CSA). General use (GU) cement is most commonly used in producing concrete for pavement construction. High-early strength (HE) cement is beneficial when a construction project must be fast-tracked. Moderate sulphate resistant and high sulphate resistant (MS and HS) are not normally in PCC pavements. Sulphate resistant mixes are most commonly used for underground installations (footings, etc) that may be in contact with native soil containing water soluble sulphates. [CSA 2003] Cementitious Materials Compendium provides further information on cement types and characteristics. Table 4.6 summarizes the types and applications.

Supplementary cementing materials are usually termed pozzolanic materials and are products which will combine with calcium hydroxide (a product of hydration of Portland cement). Pozzolanic materials include naturally occurring pozzolans (e.g., volcanic ash, diatomaceous earth), fly ash, ground blast-furnace slag, and silica fume. These materials are frequently used to improve mix workability, reduce the heat of hydration and to improve the impermeability and durability of the hardened concrete.

Table 4.6 Types and uses of Portland cement and Portland-limestone cement

Portland Cement	Portland-limestone Cement	Application
GU	GUL	For use in general concrete construction when the special properties of the other types are not required
HE	HEL	For use when high early strength is required
LH	LHL	For use when low heat of hydration is required
MS	-	For use in general concrete construction exposed to moderate sulphate action, or when moderate heat of hydration is required
HS	-	For use when high sulphate resistance is required
MH	MHL	For use in general concrete construction when moderate heat of hydration is required

4.8 Behaviours of Flexible and Rigid Pavements

Many subgrade materials have poor load carrying capabilities over a range of moisture conditions and many are sensitive to low temperature effects and experience severe reductions in load carrying capabilities when they thaw. One of the broad functions of a pavement structure is to effectively disperse loads from vehicles across these subgrade materials. Ideally, dispersing these loads results in low distributed loads on the subgrade that do not deform the material.

In general, both asphalt concrete and Portland cement concrete pavement structures serve this function, however their material characteristics cause them to behave somewhat differently. Figure 4.6 illustrates the load distribution through both a flexible and a rigid pavement type under a wheel load. Due to the rigid material's higher stiffness, the load is distributed over a greater surface area, which results in a lower distributed load on the subgrade material. The pavement structures for the two materials are generally different, but the figure illustrates the difference based on equivalent layer thicknesses.

As the figure makes clear, increasing the thickness of any of the pavement layers will serve to further reduce the magnitude of the load on the subgrade material. In general, the main component of a pavement design is determining the thicknesses of the various layers throughout the pavement structure.

The distribution of load in this way results in tensile forces at the bottom face of the surface material. Shear forces are also distributed throughout the pavement section.

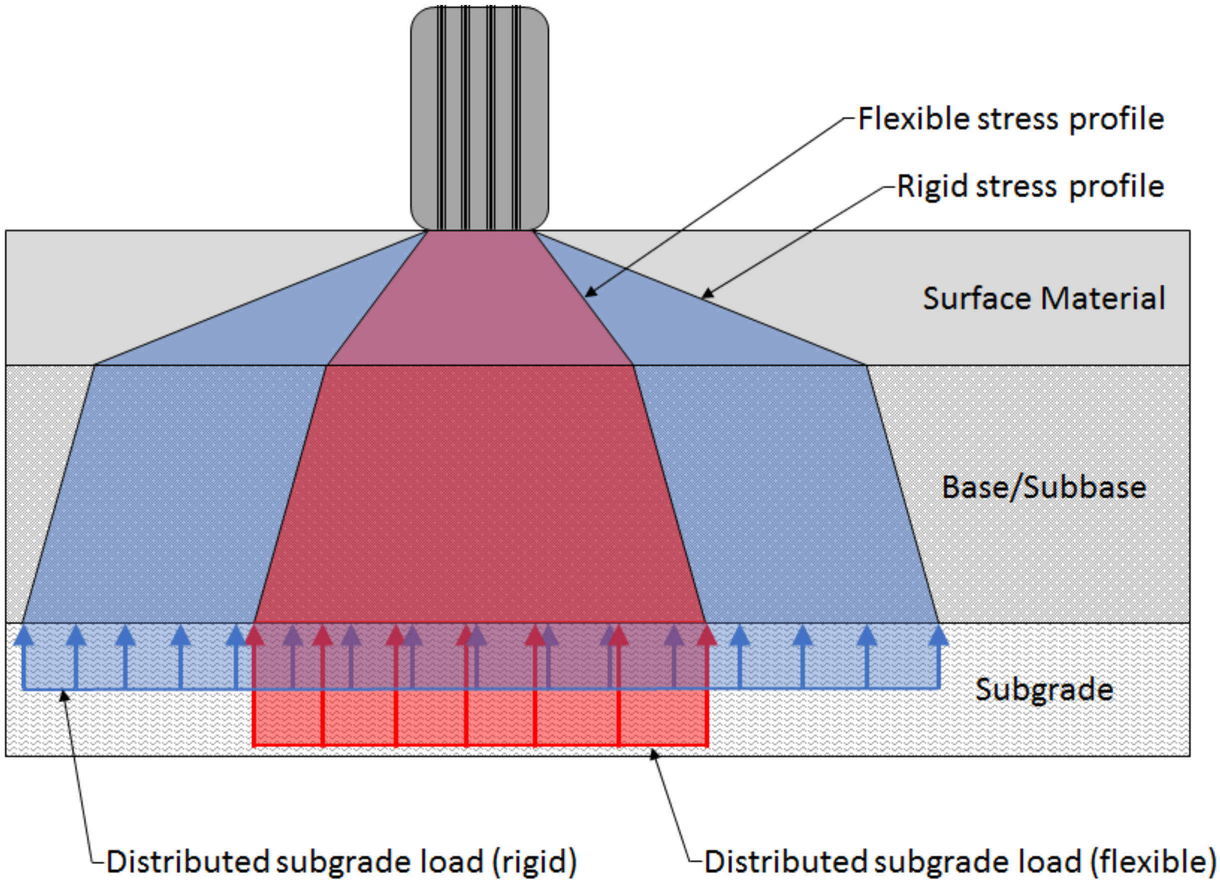


Figure 4.6 Load distribution for flexible and rigid pavement types

The different behaviours of concrete and asphalt pavements necessitate different design considerations. Joint design is a significant aspect of concrete pavement design in order to ensure that the material distributes loads in a predictable and consistent manner. Improper joint design can result in cracks which change the structural behaviour of the pavement slabs and can lead to premature failure. In asphalt pavement, the material is placed in continuous lifts. No joints are typically designed.

Asphalt pavements have tensile capacity, whereas concrete pavements have little relative to its compressive strength. For this reason, reinforcement is often included in concrete pavements to arrest the propagation of cracks that form as a result of traffic and temperature loads.

Four typical alternative designs for concrete pavements are shown in Figure 4.7.

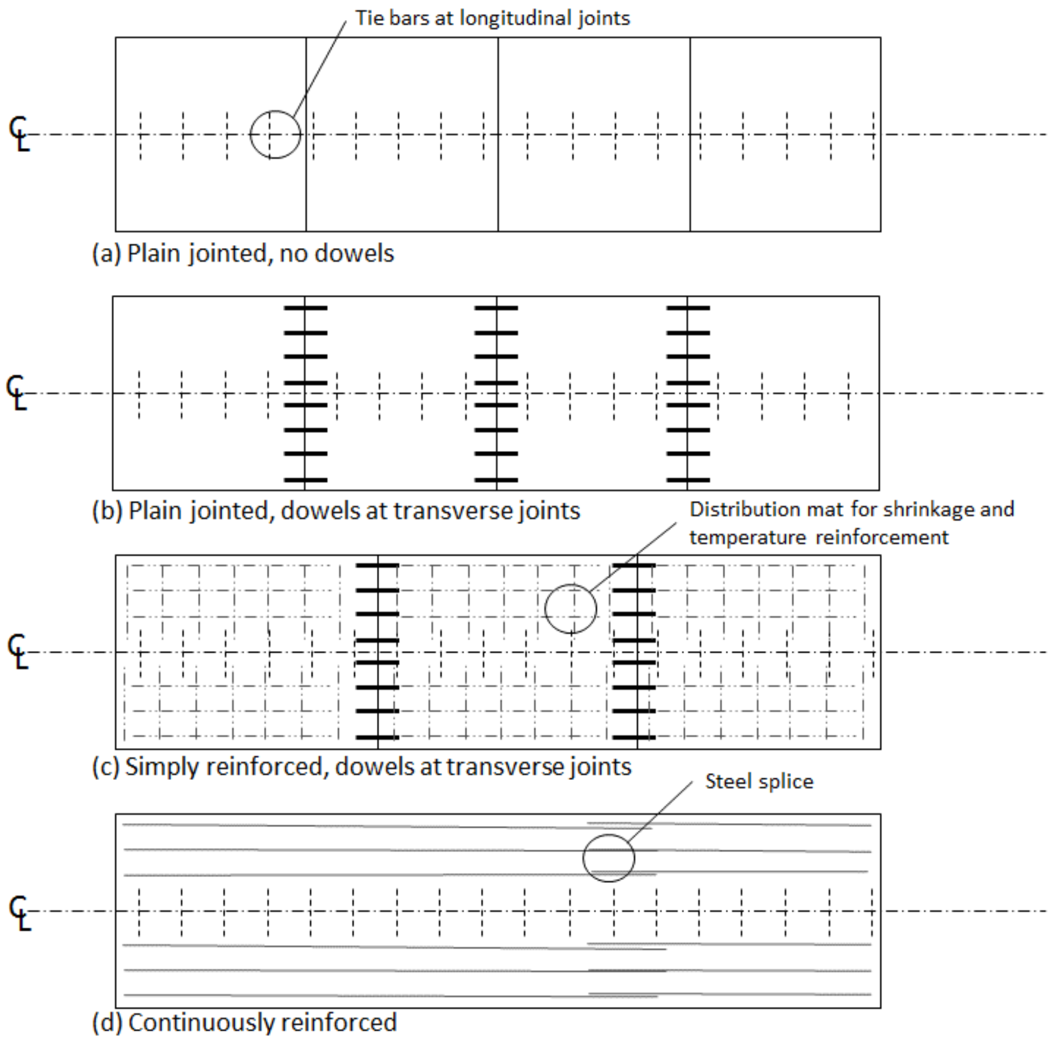


Figure 4.7 Examples of Rigid Pavements

In Figure 4.7, tie bars at longitudinal joints are typically deformed steel that bond with the concrete to hold adjacent sections together while providing shear capacity. Dowel bars at transverse joints also provide shear capacity but are smooth to allow for relative movement between adjacent slabs. This allows slabs to move under temperature-induced shrinking and swelling without causing severe damage at joints.

4.9 Pavement Deterioration and Pavement Performance

Load-associated and environmentally induced pavement deterioration eventually results in distortion of the pavement surface, which in turn influences the ride quality provided to vehicles using the system. Poor ride quality is annoying to motorists, may create physiological damage (kidney damage in truck drivers), damage cargoes, and increase vehicle operating and maintenance costs.

Pavement engineers developed the concept of pavement performance to capture the effects of pavement deterioration. The related AASHTO definitions are:

- Serviceability: the ability at the time of observation of a pavement to serve high-speed, high volume automobile and truck traffic.

- Serviceability Rating: the mean value of the independent subjective ratings by members of a special panel as to the serviceability of a section of highway.

- Performance: the trend of serviceability with load applications and age in service.

In Canada, the serviceability of a pavement is usually measured by the Riding Comfort Index (RCI) which reflects on a 0 – 10 scale the ride quality. When a highway pavement is first constructed its riding quality is usually between 8 and 9. As a pavement ages and becomes rougher the RCI decreases and pavements on arterial roads are usually resurfaced when the RCI reaches 4.5.

The RCI of pavement sections is a subjective measure of the opinions of a panel of road users about the quality of a pavement. Highway agencies normally develop statistical relationships between RCI and pavement roughness measured by some device. Pavement surface condition is then measured on a routine basis with automated collection equipment and converted to RCI for each section.

Figure 4.8**Error! Reference source not found.** illustrates a number of RCI versus Age histories that might result from a range of pavement strategies. For example, alternative A reaches an unsatisfactory RCI magnitude after about 11 years, is resurfaced with a 90 mm overlay which extends its life to about 26 years. In contrast, alternative E is constructed from very high quality (and expensive) materials and has still not reached an unsatisfactory RCI after 30 years of service. The appropriate pavement strategy will be that which minimizes to long-run life cycle costs.

It has been pointed out earlier that the principal manifestations of pavement deterioration are increasing roughness, load and non-load associated cracking of the surface, and rutting. For a well-designed pavement, rutting is caused mainly by the permanent deformation of the surface courses and the subgrade, while load-associated surface cracking is associated with excessive tensile strains induced in the underside of the upper pavement layers, and/or excessive tensile strains at the surface when very high tire pressures are involved. Low temperature-induced transverse cracking of pavements also occurs in many areas of Canada and this also contributes significantly to pavement deterioration.

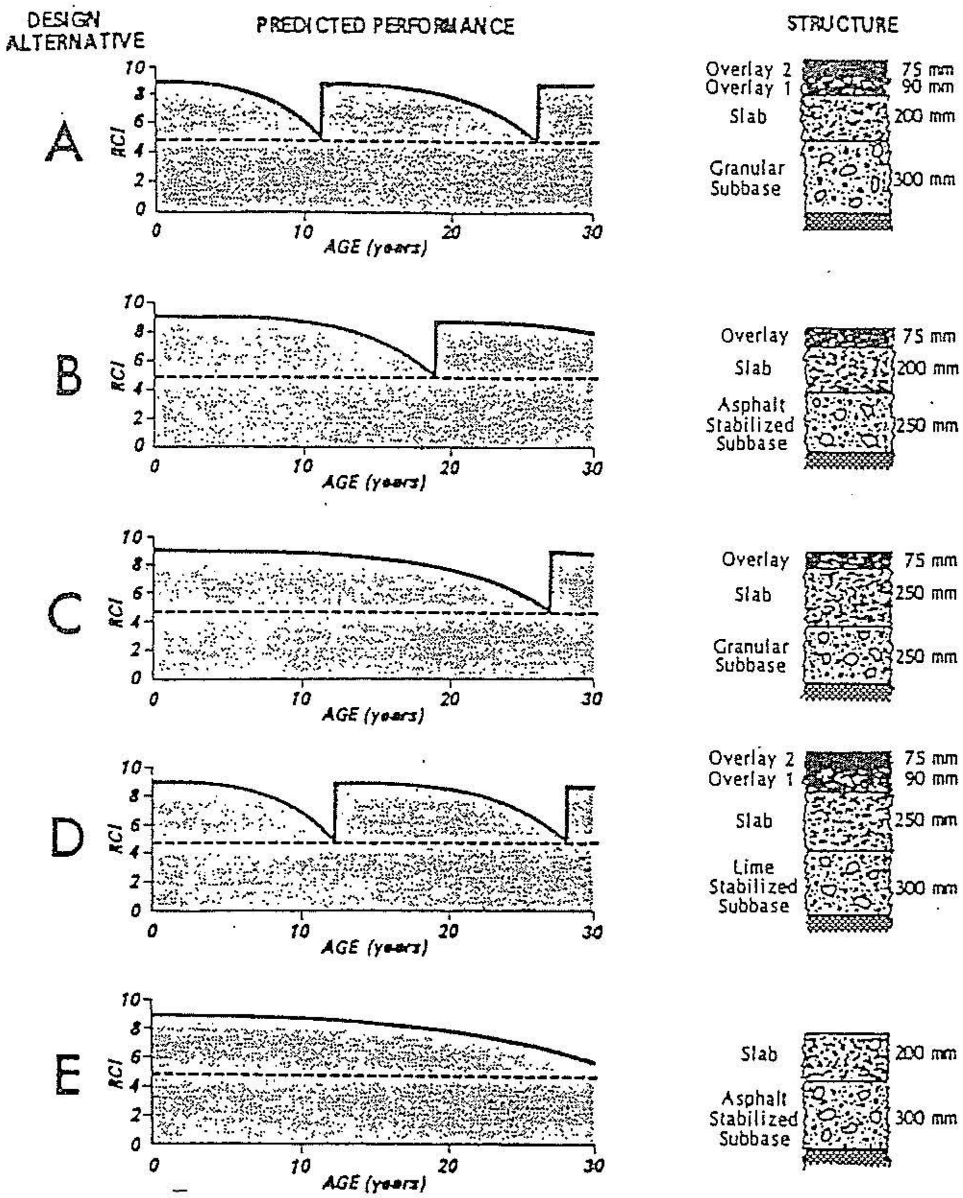


Figure 4.8 RCI vs Age Histories for Alternative Pavement Strategies

5. Pavement Structural Design

CHAPTER FIVE

5.1 Types of Structural Design Methods

Methods of pavement structural design may be classified as follows:

1. Experience-based methods, using standard sections.
2. Empirical methods in which relationships between some measured pavement response, usually deflection, or field observations of performance, and structural thickness are utilized.
3. Theory-based methods, using calculated stresses, strains, or deflections. These are sometimes called mechanistic-empirical methods [TAC 97].

The procedures based on standard sections reproduce past "successful" designs which are expected to work again. In other words, standard layer thicknesses are provided for various conditions of traffic, types of subgrade soil, class of highway, drainage, etc. With this approach, a major limitation is that experience, which relies on the interpretation of past events, cannot be easily extrapolated to other or future traffic volumes, new materials, different environmental conditions, different construction and maintenance practices, and so on.

Empirically-based procedures often use the results of measured response, such as deflection, on different pavement structures to establish limits (i.e., maximum rebound deflection) for "successful" pavements for various volumes of traffic. Most of the limitations of the standard section approach also apply to this approach.

Theory-based methods relate calculated values of stress, strain or deflection at some point(s) in the pavement structure to observed performance under various conditions, or to axle load repetitions to the end of the pavement's service life. Full-scale road tests and experimental field sections have been most useful to the development of theory-based procedures. The results of the Long Term Pavement Performance (LTPP) studies of the Strategic Highway Research Program contribute substantially. This approach has the advantage of considerable flexibility because of:

(a) its theoretical basis,

(b) the capability of using past experience for "calibration" purposes

(c) the potential for transfer from one environment to another.

However, it should be noted most theory-based design methods still have some empirical components and require additional work to make them consistent, reliable, and routine [TAC 97].

It may be desirable in certain design situations to check for excessive structural damage in terms of fatigue cracking, rutting, and low-temperature cracking. A large amount of research has been carried out to develop models for this purpose, the most recent being the Superpave procedures and software developed for the Strategic Highway Research Program [AASHTO 94, Kennedy 93].

There are a number of mechanistic-empirical design methods including those developed by Shell [Shell 1997] and the Asphalt Institute [AI 1982], and in Canada the Ontario Pavement Analysis of Costs (OPAC) method [MTO 2002]. However, the most comprehensive one is that developed under NCHRP Project 1-37A [NCHRP 2004] and known as the Mechanistic Empirical Pavement Design Guide (MEPDG) and its associated software application, DARWin-ME [AASHTO 2011]. There are a number of road agencies that are planning to implement the MEPDG method. About 15% of the provincial/federal/territorial agencies have already used the MEPDG on a limited basis and 54% intend to use it within the next five years [Tighe 2010]. TAC has established working groups to assist in the implementation of the MEPDG for Canadian agencies.

The thickness design methods described in this chapter inherently contain some built-in “calibrations” to the problems of frost heave, differential frost heave, spring strength loss, interactions of materials with the environment, and other climatic effects.

Figure 5.1 is a flow chart illustrating in a general way how a design method functions. Alternative designs would normally comprise flexible, rigid and composite types. While the choice of type might ideally be purely “competitive”, in terms of technical and life cycle economic considerations, many agencies actually do this on a policy or preference basis. The actual inputs for any particular design method may or may not include all of those listed in Figure 5.1.

5.2 General Factors Influencing Pavement Design

The following factors influence almost all pavement designs to a greater or lesser degree:

1. Traffic loading (volumes, growth rates, axle loads and distribution, tire pressures, vehicle suspension characteristics, etc.)

2. Environmental conditions (precipitation, moisture in pavement layers, temperature ranges, freeze-thaw cycles)

3. Subgrade soil (type, moisture content, properties)

4. Available or proposed pavement structure(s) (materials, material properties, unit costs, thickness ranges, specified or implicit level of reliability, etc.)

5. Expected “quality” of construction and maintenance

6. Constraints (maximum available funds, minimum initial service life to first overlay, minimum acceptable level of ride quality, etc.).

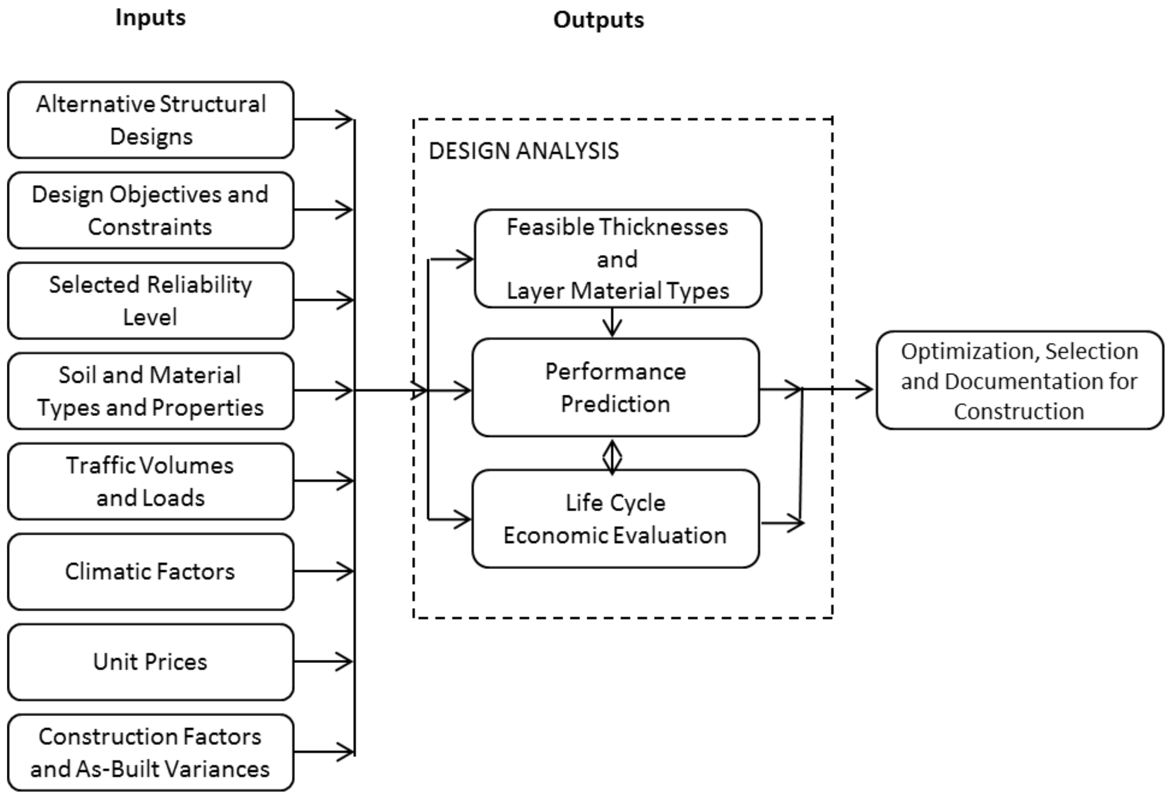


Figure 5.1 Functional scope of pavement design

Flexible pavements have often been characterized as a three (or more)-layer system, such as exemplified in the Shell design procedure [Shell 78, 92]. Three layers were originally used because of the desirability of keeping the number of design charts and graphs within practical limits. The pavement model used with a standard wheel loading is shown in Figure 5.2. In his model the subgrade, the base layer(s), and a top layer representing all asphalt layers, are characterized by Young's modulus of elasticity, E Poisson's ratio, μ and thickness, h. Analysis of stresses and strains induced in the pavement layers by traffic loads is usually considered to be critical at the underside of the asphalt layer and the top of the subgrade.

Common alternative terminology used in the AASHTO Guide [AASHTO 93] is depicted in Figure 5.3 for typical rigid or flexible pavement sections.

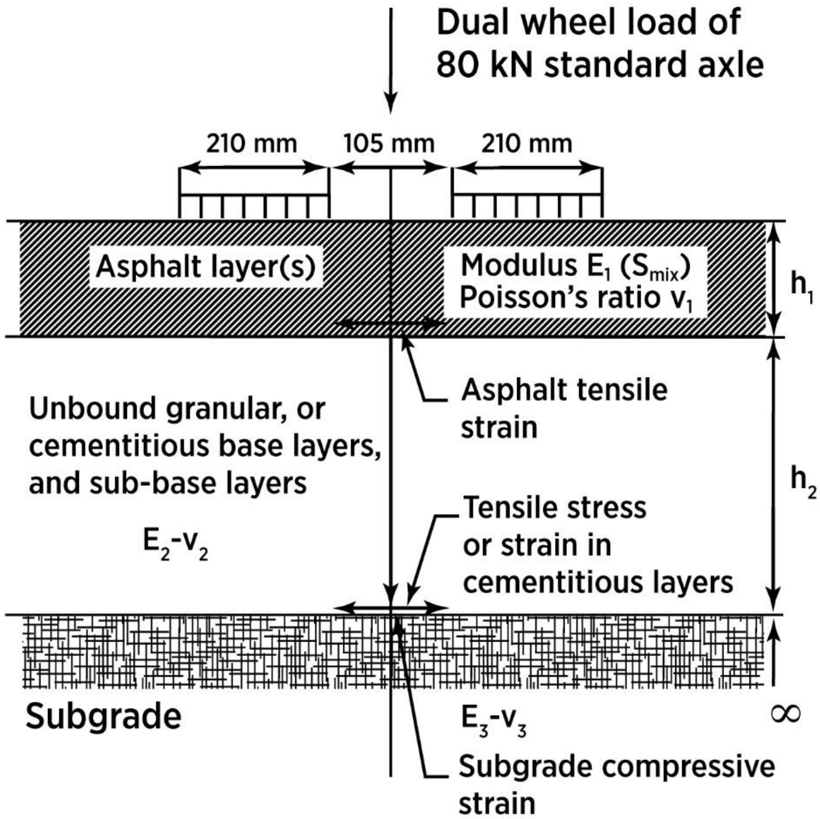
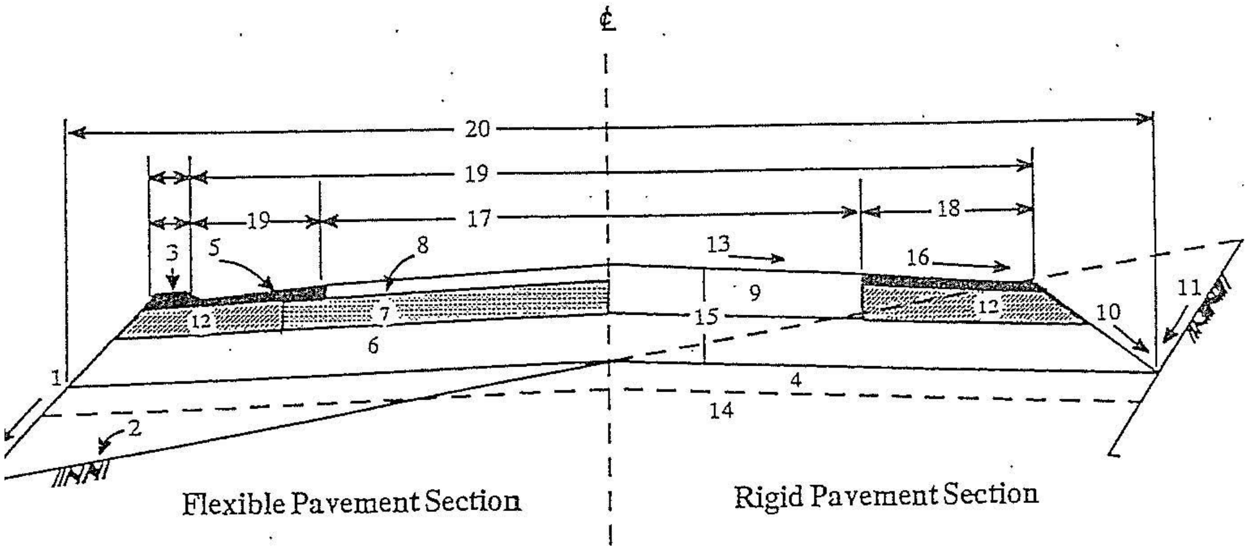


Figure 5.2 Three-layer model for a flexible pavement structure [Shell 78, 92]

5.3 Traffic Load Input to Design

In the programming, design, construction, maintenance and rehabilitation of pavements, it is desirable to think of the progressive deterioration over the pavement life-cycle in terms of traffic load and environmental causes. While the relative and absolute effects of each factor are difficult to assess in any case, it is important to have a good estimate of traffic loadings from the time a pavement is put into service through the end of the anticipated overlay or other type of rehabilitation period. Most design methods incorporate a prediction of the number of load repetitions, and the concept of equivalent standard axle loads for this purpose has been widely adopted.

As well, most agencies have well established procedures for obtaining traffic volumes and vehicle classification counts and for measuring axle loads at either fixed or portable weigh-scale sites, or from “weighing in motion” measurements.



Structural Design Terms

1. FILL SLOPE
2. ORIGINAL GROUND
3. CURB
4. SELECTED MATERIAL OR PREPARED ROADBED
5. SHOULDER SURFACING
6. SUBBASE
7. BASE COURSE
8. SURFACE COURSE
9. PAVEMENT
10. SIDE SLOPE
11. BACK SLOPE
12. SHOULDER BASE COURSE
13. CROSSFALL
14. SUBGRADE
15. PAVEMENT STRUCTURE
16. SHOULDER SLOPE
17. TRAVEL LANES
18. SHOULDER
19. ROADWAY
20. ROADBED

Figure 5.3 Typical section for rigid or flexible pavement structure [AASHTO 93]

It is beyond the scope of this Guide to present procedures for conducting traffic and vehicle classification counts, and for weighing axle loads. Rather, the purpose is to document the methods used to calculate equivalent axle loads for design purposes. This is an area of considerable importance for designing new pavements and rehabilitation and for evaluating the effects of increased legal axle load limits.

5.3.1. Load Equivalency Factor (LEF) and Equivalent Single Axle Load (ESAL)

Among the factors influencing pavement performance, loading is one of the most important. A substantial amount of research on load equivalency factors was completed as a part of the AASHTO Road Test [AASHTO 93]. Once the specific axle groups and maximum loadings for each vehicle are know, it is necessary to convert each axle group to an equivalent "standard" axle. This standard axle is a single axle with dual tires (Figure 5.4) with a total load of 18,000 lb or 80 kN. Figure 5.4 also shows different tire-axle configurations. The single axle with single tire is used in small truck and as truck steering axle for all truck types.

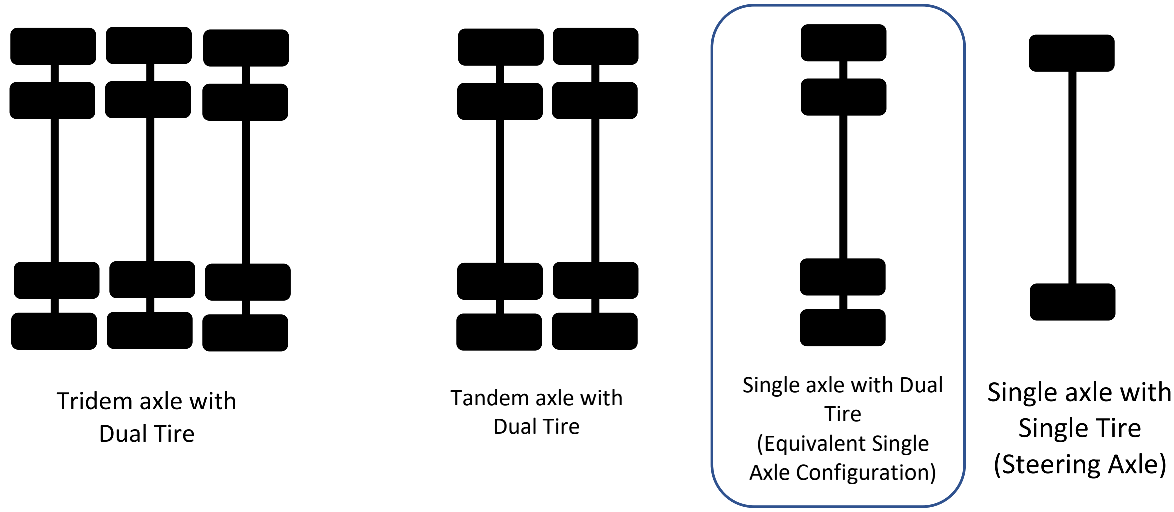


Figure 5.4 Tire-Axle Group Configurations

$$Load\ Equivalency\ Factor\ (LEF) = \left[\frac{specific\ axle\ group\ load}{standard\ axle\ group\ load}\right]^4 \quad \textit{Equation 5-1}$$

The Load Equivalency Factor, LEF, represents the number of equivalent standard axle loads per axle group having a different configuration and/or load than the standard axle (which is 18,000 lb or 80 kN in North America). This value is usually referred to as Equivalent Single Axle Loads or **ESAL**. Equation 5-1 is a simple equation to approximately find the Load Equivalence Factor for a specific axle group load in comparison to the standard axle group load. This equation is often referred to as the "Fourth Power Law" as it uses an exponent of 4. This exponent is an approximation (actually it was about 3.8 for flexible pavements as determined from the AASHO Road Test results) and varies with structural, serviceability and loading factors.

Example: Equivalent Single Axle Loads

Convert a 50 kN axle load into the number of equivalent single axle loads, where 80 kN represents the standard axle load.

Unfortunately, ESAL determination is not as simple as outlined above. The load equivalency factors are a function of axle group type, pavement type, thickness, and serviceability. AASHTO differentiates between “rigid” and “flexible” ESALs as the loss of serviceability for the passage of an ESAL was different at the AASHO Road Test for the two basic pavement types. When converting traffic to ESALs for a rigid pavement and a flexible pavement considered to be of equal load capacity, the resultant rigid ESALs are usually greater than the flexible ESALs. This does not mean that the flexible pavement cannot carry as many ESALs as the rigid pavement. It is simply how the AASHTO Guide converts the same traffic for both pavements to a number which results in the same loss of serviceability. Generally, 1 flexible ESAL is approximately equal to 1.5 rigid ESALs. This ratio is not constant, however and fluctuates with pavement design conditions.

In essence, an ESAL is a measure of damage induced by an axle and not simply a count of the number of axles or vehicles that traverse a given pavement. Flexible and rigid pavement load equivalency factors (LEF’s) can be found in the 1993 AASHTO Design Guide [AASHTO 93] and computer program DARWin-2 [AASHTO 93].

The commonly used single axle load of 18,000 lbs (8165 kg), as employed by AASHTO, is in terms of mass. Converting to force units it is 18-kips (80.1 kN). Hence, equivalent single axle loads are often expressed as $ESAL_{80}$ (in S.I. units), as used in Canada, or $ESAL_{18}$ (in Imperial units).

Load spectra is another option for designers attempting to characterize the expected traffic for a given pavement. The method characterizes loads directly using number of axles, configuration, and weight. The method doesn’t require that the loads be converted into any equivalent loads, as the actual loads are designed for. The calculations involved in designing with load spectra are generally more complex than those for ESALs (Pavement Interactive, 2008). Figure 5.5 shows the classification of small vehicles and trucks proposed by the federal Highway Administration FHWA.

5.3.2. Canadian Vehicle Weights and Dimensions Study

The Canadian Vehicle Weights and Dimensions Study [RTAC 86] was carried out to provide a sound technical basis for heavy vehicle classification and to quantify pavement damage caused by heavy vehicles. While a significant amount of research effort was expended at the AASHO Road Test in the early 1960’s, the limited number of pavement types, traffic loadings and environment resulted in speculative extrapolation of the AASHO Road Test data. In the Canadian study, fourteen test sites were instrumented to measure pavement strain and deflection under various configurations (axle loads, axle groups, and axle spacings) of a heavy test vehicle. The test sites were located in the five major geographic regions of Canada and the testing was carried out using single, tandem, and tridem axles at a range of loadings to determine the potential pavement damage in terms of Load Equivalency Factors (LEF).

The overall average LEFs for the Canadian study are shown in Figure 5.6. Regression equations for the relationships are also shown in Figure 5.6.

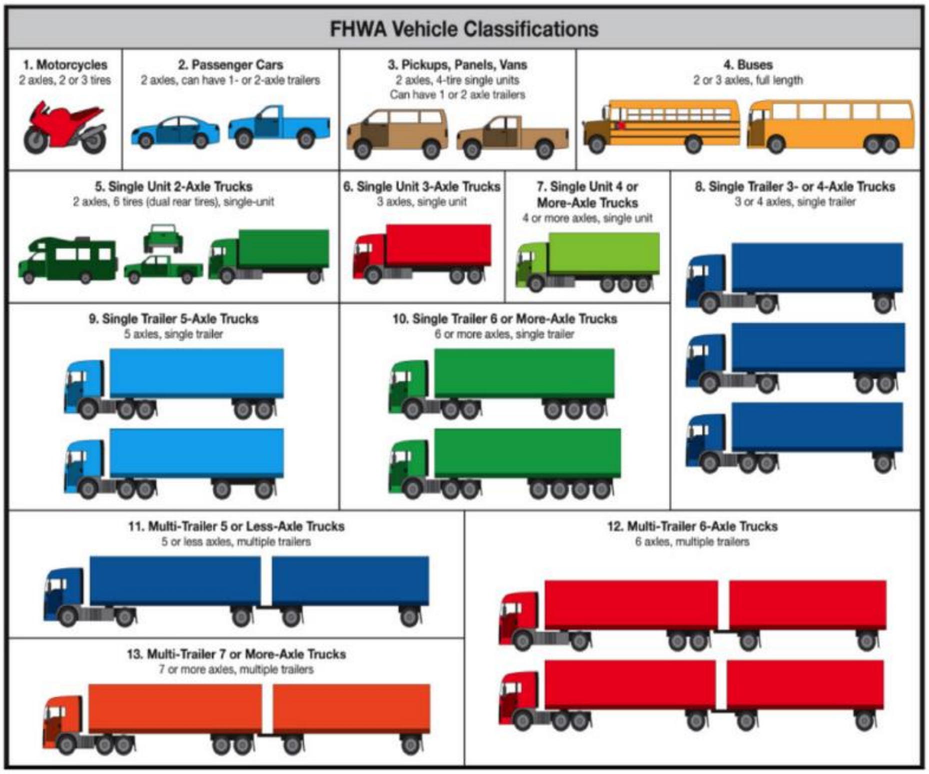


Figure 5.5 FHWA Truck classifications based on Axles and Vehicles [FHWA U.S. DOT 2012]

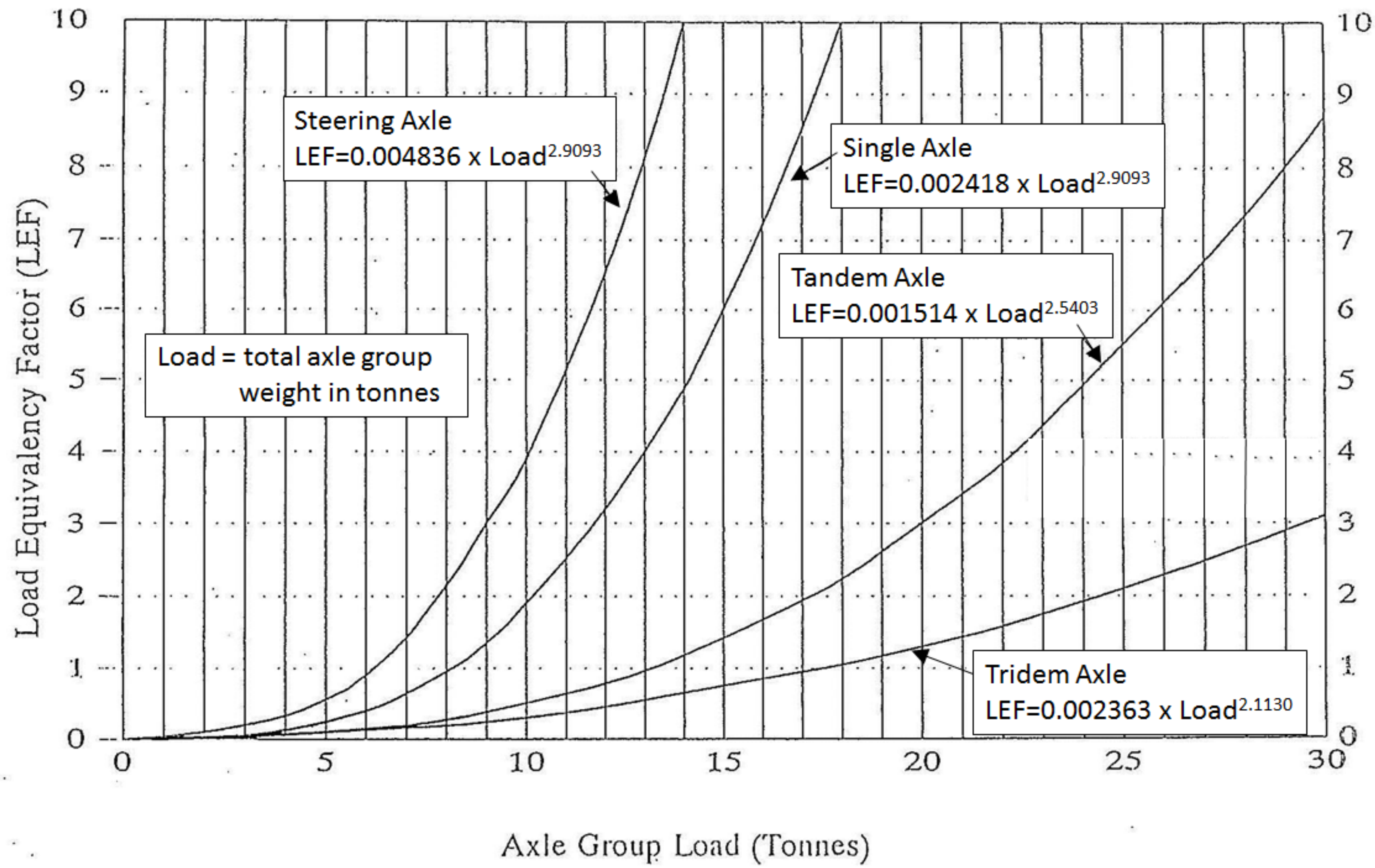


Figure 5.6 Average load equivalency factors from the Canadian Vehicle Weights and Dimension Study [RTAC 86]

5.3.3. Modified Asphalt Institute Method

A modified Asphalt Institute method has been developed to utilize the truck factor approach while simplifying the input procedure for pavement design engineers. In this method, the following general equation is used:

Equation 5-2

where:

ESAL = Equivalent Single Axle Loads per Lane per Year
AADT = Average Annual Daily Traffic (all lanes, both directions)
HVP = Heavy Vehicle Percentage (divided by 100)
HVDF = Heavy Vehicle Distribution Factor (% of heavy vehicles in the design lane)
TF = Number of equivalent axle loads per heavy vehicle (Truck Factor)
TDY = Traffic Days per Year

Typically, most pavements are constructed with all lanes having the same pavement structure. Trapezoidal construction, where the lane accommodating most of the heavy vehicle traffic is constructed to a higher structural capacity, is not common practice.

- The lane subjected to the heaviest vehicle traffic is selected as the design lane.

The values of the Truck Factors (TF) proposed by the Asphalt Institute are given in Table 5.1.

Table 5.1 Distribution of Truck Factors (TF) For Different Classes of Highways and Vehicles [TAI 91a]

TABLE IV-5 DISTRIBUTION OF TRUCK FACTORS (TF) FOR DIFFERENT CLASSES OF HIGHWAYS AND VEHICLES—UNITED STATES*

Vehicle Type	Truck Factors											
	Rural Systems						Urban Systems					
	INTER-STATE	OTHER PRINCIPAL	MINOR ARTERIAL	COLLECTORS MAJOR	COLLECTORS MINOR	RANGE	INTER-STATE	OTHER FREEWAYS	OTHER PRINCIPAL	MINOR ARTERIAL	COLLEC-TORS	RANGE
Single-unit trucks												
2-axle, 4-tire	0.003	0.003	0.003	0.017	0.003	0.003-0.017***	0.002	0.015	0.002	0.006	---	0.006-0.015***
2-axle, 6-tire	0.21	0.25	0.28	0.41	0.19	0.19-0.41	0.17	0.13	0.24	0.23	0.13	0.13-0.24
3-axle or more	0.61	0.86	1.06	1.26	0.45	0.45-1.26	0.61	0.74	1.02	0.76	0.72	0.61-1.02
All single-units	0.06	0.08	0.08	0.12	0.03	0.03-0.12	0.05	0.06	0.09	0.04	0.16	0.04-0.16***
Tractor semi-trailers												
4-axle or less	0.62	0.92	0.62	0.37	0.91	0.37-0.91	0.98	0.48	0.71	0.46	0.40	0.40-0.98
5-axle**	1.09	1.25	1.05	1.67	1.11	1.05-1.67	1.07	1.17	0.97	0.77	0.63	0.63-1.17
6-axle or more**	1.23	1.54	1.04	2.21	1.35	1.04-2.21	1.05	1.19	0.90	0.64	---	0.64-1.19
All multiple units	1.04	1.21	0.97	1.52	1.08	0.97-1.52	1.05	0.96	0.91	0.67	0.53	0.53-1.05
All trucks	0.52	0.38	0.21	0.30	0.12	0.12-0.52	0.39	0.23	0.21	0.07	0.24	0.07-0.39

***Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.*
*****Including full-trailer combinations in some states.*
******See Article 4.05 for values to be used when the number of heavy trucks is low.*

Example: Flexible Pavement Design

A flexible pavement is to be designed for the reconstruction of a 10 km section of the two-lane Highway 101 (RAU-90) in Northern Ontario between Chapleau and Foleyet. Available traffic information reveals that the average annual daily traffic for the section of the highway 2,500 vehicles. Forty percent of these vehicles are passenger cars. The distribution of trucks is expected to be as follow:

Single-unit trucks
2-axle, 4-tire = 47 percent
2-axle, 6-tire = 17 percent
3-axle or more = 12 percent

Multiple-unit trucks
4-axle or less = 6 percent
5-axle = 18 percent
6-axle or more = 0

Determine the first year Design ESAL for the design lane.

The estimated future traffic of a section is a function of the traffic level, the length of the performance period, and the rate of traffic growth. Generally, an ESAL estimate for the first year is calculated using an equation similar to Equation 5-2. The performance period and the estimated growth rate are then used to calculate the total number of ESALs that the pavement will support over the design period. Equation 5-3 can be used to make these calculations.

Equation 5-3

where:

- g = the projected growth rate over the performance period (%)
- t = the length of the performance period (years)

Example: Flexible Pavement Design (continued)

After the estimate was made for the first year of Highway 101 in the previous example, it was decided that the new pavement would be designed for a 20 year life span. The designer needs an estimate of how much traffic loading can be expected during that period. A transportation analyst estimates that based on certain projections, the annual growth rate in traffic could be anywhere from 0% - 2%/year for the next 20 years. What is the range of total ESALs that these two projections imply?

5.4 Design Methods for Conventional Flexible Pavements

5.4.1. Standard Sections

A very useful part of the RTAC Guide [RTAC 77] was the summary table of then current structural thicknesses which were being used by various agencies for “average conditions”. These tables were indicative only and not considered as precise; however, they did serve as an indicator of the probable range of values which would be used under such average conditions. In effect, they represent a catalogue of experience-based design.

Typical structures were presented for two classes of subgrade strength and three levels of traffic. The subgrade descriptions and strengths were meant to be illustrative and did not take into account local environmental conditions or variations in soil type. The structures given were for provincial highway agencies and did not reflect special considerations of traffic, drainage, etc. that may be applicable to urban pavement conditions.

A survey of provincial agency practices, undertaken for an earlier version of the TAC PADMG has provided information on typical conventional (granular base) pavement structures for three levels of subgrade support and three levels of traffic. The “weak” subgrade could be considered as a lacustrine clay, with the “medium” subgrade a glacial till. The third level of subgrade support was considered as “strong” to reflect those situations where the subgrade could consist of a clayey gravel, or essentially granular material. Cumulative ESALs of 0.5, 1.0 and 10×10^6 were selected for the three levels of traffic loads.

It may be noted that some agencies, such as Saskatchewan for example, would not build on the “weak” subgrade without first treating it for swelling. They have found, as have other agencies, that the design problem is not so much one of structural capacity but of limiting expansion of the subgrade at cracks. A summary of the information collected for the three traffic levels is given in Table 5.2, Table 5.2, and Table 5.3.

Table 5.5 lists structural equivalencies which various agencies have used for different layer materials. This table was developed from those reported in the RTAC Guide [RTAC 77] and does not include all agencies at the present time. The values represent guidelines for comparison purposes.

It should be noted that there is a wide range of equivalencies. Therefore, they should not be transferred from region to region. Equivalencies listed have been found to work by a specific agency in a specific region for a relatively narrow range of traffic conditions and structural component thicknesses.

The total GBE is calculated by summing the actual layer thicknesses times their respective ratio given for the component layer type.

Table 5.2 Typical pavement thicknesses (mm) used by provincial agencies for selected subgrade and traffic loading.

Subgrade Type	Conventional Pavement Structure Course	BC	AB	SK	MB	ON	QC	NB	NS	PEI	NF	YT**	PW*** & GSC
"Weak"	Asphalt concrete	75	80	Seal*	85	90	85	140	100		80	100	50
Lacustrine Clay	Asph. stab. gran.		50										
CBR ~ 3.0	Granular base	300	300	150	150	150	150	150	150		100	150	150
Group Index ~ 20	Granular subbase			470	300	450-600	450-600	450	250	n/a	200		200
Unified Soil	Select granular	300										450	
Class.-CH						690-840	685-835						
	Total	675	430	620	535			740	500		380	700	400
"Medium"	Asphalt concrete	75	80	Seal**	85	90	85	140	100		80	100	50
Glacial Till	Asph. stab. gran.		50										
CBR ~ 5	Granular base	300	200	150	100	150	150	150	150		100	150	150
Group Index ~ 10-12	Granular Subbase			320	200	450	450-600	450	200	n/a	200		150
Unified Soil	Select granular	300										300	
Class.-CL													
	Total	675	330	470	385	690	685-835	740	450		380	550	350
"Strong"	Asphalt concrete	75	60	Seal**	85	90	85	140	100		80	100	50
Clayey Gravel	Asph. stab. gran.		50										
CBR ~ 20	Granular base	300	150	200	100	150	150	150	150		100	150	150
Group Index ~	Granular subbase				100		300-450	450	200	n/a	200		
Unified Soil	Select granular	0-300										150	
Class.-GC													
	Total	375-675	280	200	285	240	535-685	740	450		380	400	200

* A double seal on base would be used as the surface for up to 1500 AADT

** Yukon has alternative designs of Bituminous Surface Treatment (BST) on 200 mm of granular base on 450, 300 and 150 mm of select granular, respectively, for the three subgrade types

*** Public Works and Government Services Canada

Table 5.3 Typical pavement thicknesses (mm) used by provincial agencies for selected subgrade and traffic loading.

Subgrade Type	Conventional Pavement Structure Course	BC	AB	SK	MB	ON	QC	NB	NS	PEI	NF	YT*	PW** & GSC
"Weak"	Asphalt concrete	100	100	100	100	130	105	140	150		120	100	100
Lacustrine Clay	Asph. stab. gran.		50										
CBR ~ 3.0	Granular base	300	300	150	175	150	200	150	150		150	150	200
Group Index ~ 20	Granular subbase			360	350	450-600	525-675	450	300		300	150	450
Unified Soil	Select granular	300								n/a	500	450	
Class.-CH													
	Total	700	450	610	625	730-880	830-980	740	600		1070	850	750
"Medium"	Asphalt concrete	100	90	80	100	130	105	140	150		120	100	100
Glacial Till	Asph. stab. gran.		50										
CBR ~ 5	Granular base	300	230	150	150	150	200	150	150		150	150	200
Group Index ~ 10-12	Granular Subbase			260	225	450	525-675	450	250		300	150	300
Unified Soil	Select granular	300								n/a	500	300	
Class.-CL													
	Total	700	370	490	475	730	830-980	740	550		1070	700	600
"Strong"	Asphalt concrete	100	80	60	100	130	105	140	125		120	100	100
Clayey Gravel	Asph. stab. gran.		50										
CBR ~ 20	Granular base	300	200	180	100	150	200	150	150		150	150	150
Group Index ~	Granular subbase				100	150	300-450	450	200		300	150	150
Unified Soil	Select granular	0-300								n/a	500	150	
Class.-GC													
	Total	400-700	330	240	300	430	605-755	740	475		1070	550	400

* Yukon has alternative designs of Bituminous Surface Treatment (BST) on 300 mm of granular base on 450, 300 and 150 mm of select granular, respectively, for the three subgrade types.

Table 5.4 Typical pavement thicknesses (mm) used by provincial agencies for selected subgrade and traffic loading.

Subgrade Type	Conventional Pavement Structure Course	BC	AB	SK	MB	ON	QC	NB	NS	PEI	NF	YT··	PW···· & GSC
“Weak”	Asphalt concrete	100	120	150	125	180	180	140	200			125	150
Lacustrine Clay	Asph. stab. gran.		50										
CBR ~ 3.0	Granular base	300	400	200	175	150	250	150	150			150	200
Group Index ~ 20	Granular subbase			390	350	600-800	525-900	450	500	n/a	n/a	150	600
Unified Soil	Select granular	650											
Class.-CH													
	Total	1050	570	740	650	930-1130	955-1330	740	850			875	950
“Medium”	Asphalt concrete	100	120	130	125	180	180	140	200			125	150
Glacial Till	Asph. stab. gran.		50										
CBR ~ 5	Granular base	300	230	200	150	150	250	150	150			150	200
Group Index ~ 10-12	Granular Subbase			280	225	450-600	525-675	450	300	n/a	n/a	150	400
Unified Soil	Select granular	425										300	
Class.-CL													
	Total	825	400	610	500	780-930	955-1105	740	650			725	750
“Strong”	Asphalt concrete	100	100	90	125	180	180	140	200			125	150
Clayey Gravel	Asph. stab. gran.		50										
CBR ~ 20	Granular base	300	250	230	100	150	250	150	150			150	200
Group Index ~	Granular subbase				100	300	300-450	450	200	n/a	n/a	150	300
Unified Soil	Select granular	0-300										150	
Class.-GC													
	Total	400-700	400	320	325	630	730-880	740	550			575	650

British Columbia Department of Transportation and Highways:
Design Method: AASHTO design method used but rounded off or default to minimum thicknesses as per B.C. Pavement Design Standards (as per draft Technical Circular – May, 1995). Thickness of base and subbase will depend on environment, permeability of base and strength of subgrade.

Alberta Transportation & Utilities
Design Method: Asphalt Institute (1963) Design Equations, followed by prototyping against previous projects of similar character having measured performance. Where necessary, "final pavement" depths are determined on the basis of FWD testing of the staged base, using back-calculated values for resilient moduli of the layers to establish required "overlay" depths.

Additional Comments: 1. Pre-treatment of subgrades for CH clays having a high swelling potential is uncommon and handled on a project-specific basis. Loading the subgrade (by ensuring a thicker total structure than normal), and lime treatment are two solutions suggested. 2. AT&U is currently changing its staged base construction design strategy to exclude the 50 mm layer of asphalt stabilized base course [ASBC] and substitute a 60 mm layer of asphalt concrete pavement [ACP]. The layer depths of the predicted complete structure will be adjusted on the basis of conventional equivalencies, generally to provide similar "Granular Equivalent" structures for each case, while providing adequate ACP thickness for traffic conditions.

Saskatchewan Highways & Transportation:
Design Method: Modified Shell Design.
Additional Comments: * A double seal on base would be used as the surface for up to 1500 AADT.

Manitoba Department of Highways and Transportation:
Design Method: Agency Method based on Group Index. Reference CGRA [1965].
Additional Comments: Structural equivalencies: a) asphaltic concrete = 2.0, b) granular base = 1.0, c) granular subbase = 1.0

Nova Scotia Department of Transportation:
Design Method: Asphalt Institute, 15-year Life.

Transport Quebec (MTQ):
Design Method: AASHTO 1993 (modified and adapted).
Additional Comments: Average values, Thickness may vary within the province. Except for high trafficked roads above weak soils, granular subbase thicknesses are the minimum required for frost protection.

Table 5.5 Approximate layer equivalencies used by various agencies for new pavement designs

Provincial Agency	Component Layer Material Ratio Actual Thickness = Granular Base Equivalency (GBE) **
British Columbia	2.00 asphalt concrete 1.00 well-graded gravel base 1.70 asphalt treated base course 1.10 intermediate graded base 1.20 open graded base 0.70 pit run subbase
Alberta	2.25 asphalt concrete 1.00 crushed gravel (granular base) 1.30 soil cement 1.80 asphalt treated gravel base
Saskatchewan	Considered as a variable and therefore not used.
Manitoba	2.00 asphalt concrete 1.00 gravel base 1.33 sand asphalt or soil cement
Ontario	2.00 asphalt concrete 1.80 treated base (PC) 1.50 treated base (asphalt) 1.00 granular base 0.67 granular subbase 1.00 OGDL
Quebec***	2.00 asphalt concrete 1.25 crushed rock base 1.00 gravel base or subbase 0.50 sand subbase 2.00 soil cement (150 mm or less) 1.25 soil cement (more than 150 mm) 0.75 lime stabilized clay 1.40 asphalt stabilized base
New Brunswick	2.00 asphalt concrete 1.00 crushed rock 1.00 soil cement (150 mm or over) 1.00 asphalt stabilized base 0.67 gravel subbase
PEI	2.00 asphalt concrete 1.00 crushed rock 0.67 crushed gravel 1.00 soil cement 1.00 asphalt stabilized base
Newfoundland	2.50 asphalt concrete 1.00 graded crushed rock 1.00 graded crushed gravel 0.83 gravel subbase 0.63 sandy gravel

** i.e., 1 mm of pit run subbase (B.C.) = 0.7 mm of granular base

*** The GBE's for Quebec are approximations that have been used in the past. Currently, however, Quebec considers this a variable, the same as Saskatchewan.

Example: Experience-Based Design

An example of an experience-based design will illustrate the layer thickness selection process that may be used as an initial step in a more complete design or as a final design if the scale of the project does not warrant further investigation.

Suppose a two-lane secondary rural highway constructed on a glacial till subgrade in Alberta is expected to carry approximately 800,000 equivalent single axle loads (ESALs) during a selected design period of 20 years. The following conventional flexible pavement structure might be initially considered.

Component	Thickness	Granular Base Equivalent (GBE)
asphalt concrete	100 mm	
asphalt stabilized gran. base	50 mm	
granular base	300 mm	
Total thickness of pavement structure =		Total GBE =

If granular materials were relatively costly and/or in short supply, a soil cement base might be considered. Consider an example with a soil cement base, again for the Alberta case:

Component	Thickness	Granular Base Equivalent (GBE)
asphalt concrete	100 mm	
asphalt stabilized gravel base	50 mm	
soil cement base	230 mm	
Total thickness of pavement structure =		Total GBE

It could be reasonable to assume that the chosen thicknesses of component layers are the most practical from a construction standpoint, and that any structural deficiency could be compensated for in the initial asphalt concrete surface or in a future overlay.

It should be emphasized that selecting a structural design according to the foregoing example is only a preliminary step. For a small project or relatively low volume road it might be sufficient. However, more detailed consideration is usually warranted in order to evaluate alternative possible designs or make necessary modifications to the initial, preliminary design.

5.4.2. AASHTO Flexible Pavement Empirical Design Method [AASHTO 93]

Because the AASHTO 93 principle was developed in the United States, all units in this section are Imperial (SI) units. If values are given in metric units, they should be converted to SI units before using any of the following nomographs or tables.

1” (inch)	= 25.4 mm
1 psi	= 0.0069 MPa

The procedure developed in the AASHTO Guide [AASHTO 93] for new construction or reconstruction is basically an extension of the algorithms originally developed from the AASHO Road Test. Major modifications to the original practices include the following:

1. The introduction of resilient modulus to provide a rational characterization of subgrade soil and other layer material properties.

2. The layer coefficients for the various materials are related to resilient modulus as well as California Bearing Ratio (CBR) and R-value.

3. The environmental factors of moisture and temperature are on a more objective basis and replace the subjective regional factor term previously used.

4. Reliability is introduced so that the designer can use the concept of risk analysis for various classes of roads.

Material properties for structural design are based on characterization of an elastic or resilient modulus. Laboratory resilient modulus tests (AASHTO T294) on subgrade materials should be performed on representative samples in stress and moisture conditions simulating those of the primary moisture seasons. Procedures are described in the Guide to generate an effective roadbed soil resilient modulus which is equivalent to the combined effect of all the seasonal modulus values. Alternatively, the resilient modulus values may be determined by correlations with soil properties; i.e., clay content, moisture content, plasticity index (PI), etc. The purpose of identifying seasonal moduli is to quantify the relative damage a pavement is subjected to during each season of the year and treat it as part of the overall design.

The roadbed criteria, expected load, variability, and serviceability loss are all used to determine a structural number (SN) which represents a degree of capacity that a road in the given condition must provide.

Determination of the required structural numbers (SN) involves the use of a nomograph, shown in Figure 5.7, which solves a specific design equation and parameters required for specific conditions, including:

1. estimated future traffic for the performance period,

2. the reliability, R, which assumes all inputs are at an average value,

3. overall standard deviation, S_o, of the inputs,

4. effective resilient modulus of the subgrade, M_r, and for each layer,

5. design serviceability loss, $\Delta PSI = p_o - p_i$, where PSI is Present Serviceability Index (scale of 0 to 5), p_o is the initial Serviceability Index and p_t is the terminal Serviceability Index.

The estimated future traffic of the section is a function of the traffic level, the length of the performance period, and the rate of traffic growth. This is calculated using a procedure similar to that shown in the previous section (Equation 5-3).

The reliability level of the design is chosen based on the "importance" of achieving the performance goals. Generally, this value will increase with the traffic volumes, but roads which serve critical purposes can also be designed with high reliability levels. Typical reliability levels are outlined in Table 5.6, though the reliability level should always be reviewed on a case-by-case basis. The reliability levels correspond to the percentage of time the design meets its performance goals. For instance, 90% reliability indicates that 10% of the time the design may prove to be inadequate. Higher reliability generally corresponds to thicker pavement structures.

Table 5.6 Recommended Reliability Levels (Adapted from AASHTO 93)

	Recommended Level of Reliability	
Functional Classification	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Other Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

S_0 denotes the standard deviation, or standard error, of the inputs. Largely, this error is due to the imprecise nature of forecasting traffic and pavement performance. Larger S_0 values correspond to larger variations and therefore require more conservative designs. Typically, flexible pavements have S_0 values from 0.4 to 0.5, while rigid pavements have values from 0.35 to 0.4. Reasons for a high S_0 value could include:

- imprecise traffic counting/projecting practices
- Potential for rapid population/business growth/drop in a given area
- Unpredictable climate affecting base/subbase performance
- Low-quality materials only available for pavement construction

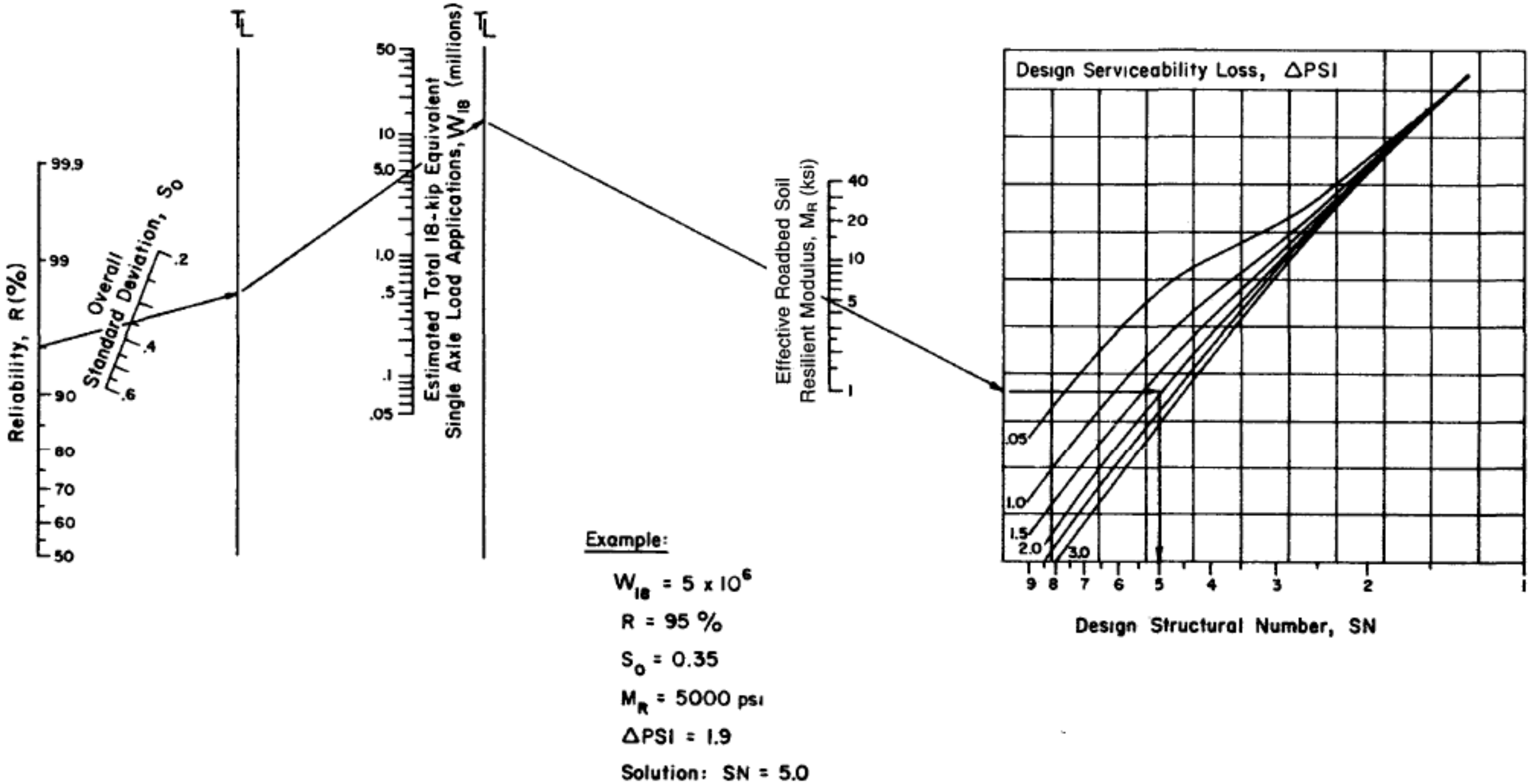


Figure 5.7 Design Nomograph from AASHTO 1993 Pavement Design Handbook (AASHTO, 1993)

The effective resilient modulus (M_R) of the materials supporting the pavement play a key role in the performance of the pavement. The resilient modulus is similar to modulus of elasticity (E) which is an important engineering property. The elastic modulus indicates the stiffness of a given material under a slowly applied load. The modulus is calculated by dividing the average stress of an element by the strain induced by this stress. Therefore, a material with a high elastic modulus (low strain under a given load) is a stiff material. Resilient modulus is a very similar material characteristic, but under the application of high frequency loads, similar to those applied by traffic. Stiffer materials (high M_R) provide better support for the pavement structure.

Climate plays a large role in the resilient modulus of a soil, especially in parts of Canada where winter and summer months are significantly different. During winter months, low temperatures will freeze the soils to the typical frost depth. This serves to stiffen the material, thereby increasing the pavement capacity. When temperatures begin to rise during spring months, the ice which is close to the surface will melt before that which is lower. This water cannot drain through the soil structure and results in soil with very high water content. This soil loses strength and the resilient modulus temporarily drops until water drainage is fully restored. This phenomenon is why many low-volume roadways have seasonal load restrictions, as the pavements will deteriorate very quickly under heavy loads with weakened subgrade supporting it.

In the case of flexible pavements, M_R values for the roadbed which are measured throughout the year are converted into an effective M_R through the use of a relative damage factor. Based on M_R, the following equation can be used to calculate the relative damage (u_f).

Equation 5-4

where:

- u_f = the relative damage for a given modulus
- M_R = the resilient modulus for a time period (psi)

The relative damage values across a year can be averaged and then converted back into an effective modulus value which can be used in the pavement design.

Serviceability loss serves to define the performance of a pavement. Present serviceability index (PSI) refers to a ride quality rating of a pavement at a given time, on a scale of 0-5. The PSI takes into account all factors that affect ride quality, including cracking, rutting, slope variability, etc. A freshly constructed pavement can generally be assumed to have an initial serviceability (p_i) of 4.2 to 4.6. The terminal serviceability (p_t) refers to the PSI rating when a pavement requires maintenance or rehabilitation. The terminal serviceability depends on the roadway type and a given agency's maintenance practices and budget.

Generally, major highways should have a terminal serviceability of 2.5, while highways with lesser traffic volume can use 2.0. Serviceability loss (ΔPSI) refers to the difference between these two values.

Equation 5-5

Using these values, the nomograph in **Error! Reference source not found.** can be used to etermine the structural number which the new pavement must have to address all of the input considered.

Example:

A pavement will be constructed in Southwestern Ontario. The pavement will be part of a rural collector system. The pavement is expected to have an initial traffic load of 500,000 ESALs with a 1.5% growth rate over the pavement's 12-year service life. The roadbed material has been tested over the past year, and the M_R values that were found are summarized in the table below. The agency is interested in learning the required structural number based on high-end and low-end reliability levels (95% and 75% reliability, respectively). The agency also estimates that the standard error for their information gathering is approximately 0.45.

Month	Measured M_R (psi)	Relative Damage (u_f)
January	20,000	
February	20,000	
March	2,500	
April	4,000	
May	4,000	
June	7,000	
July	7,000	
August	7,000	
September	7,000	
October	7,000	
November	4,000	
December	20,000	

After determining the required structural number (SN), the designer then must design a pavement structure that provides that SN, using Equation 5-6.

Equation 5-6

where:

SN = the structural number of the given pavement
a_i = the layer coefficient of layer "i"
D_i = the thickness of layer "i"
m_i = the drainage coefficient of layer "i"

Pavement layer coefficients may be based on those traditionally developed and used in the original AASHTO procedure, or more preferably derived from test roads or satellite sections. Charts are also available for estimating structural layer coefficients from various base strength parameters as well as resilient modulus test values, as shown in Figure 5.8 to Figure 5.12. These figures show the general means for determining "a" for a given layer type.

The moisture content has a significant effect on the structural capacity of base and subbase materials, and AASHTO 93 addresses this concern by incorporating drainage coefficients into the calculation of SNs. The effectiveness of the ability of various drainage methods to remove moisture from the pavement is not described with detailed criteria but uses these coefficients to account for this effect. Table 5.7 summarizes typical "m" values based on the quality of drainage and the percentage of the time that the pavement is exposed to saturated conditions.

Asphalt layers and stabilized layers (either asphalt or cement) are generally affected by moisture conditions much less than base and subbase layers. Therefore, drainage coefficients for asphalt layers are assumed to be 1 (i.e. moisture conditions do not affect the structural capacity). For the purposes of this design, this is a reasonable assumption, though both asphalt and stabilized layers are affected by moisture conditions in actuality.

Example:

What are the structural numbers (SN_i) for the following layers:

1) 4" layer of asphalt with E_{AC}=300,000 psi

2) 12" layer of base material, CBR = 60, good drainage, saturated 4% of time on average

3) 8" layer of asphalt-stabilized base, modulus = 200000psi, fair drainage, 3% saturated

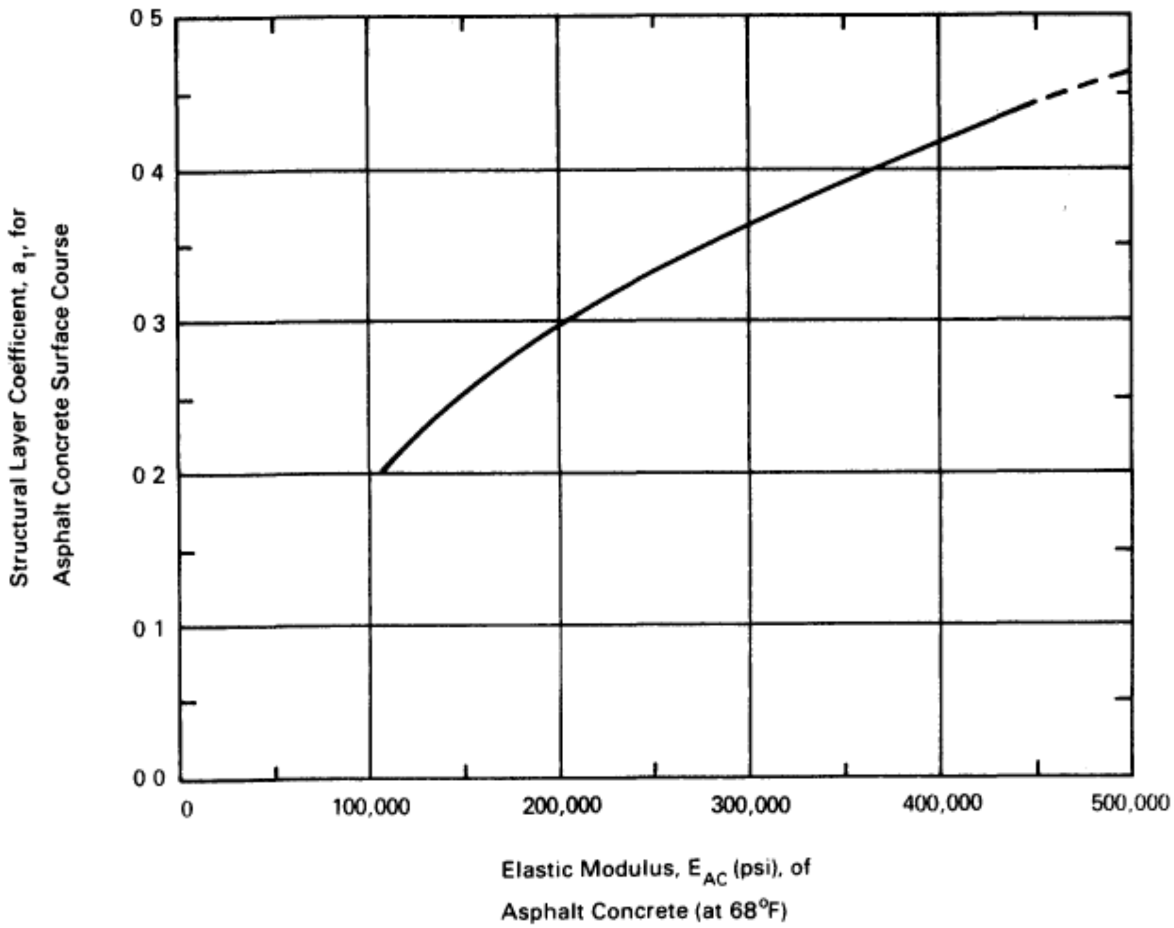
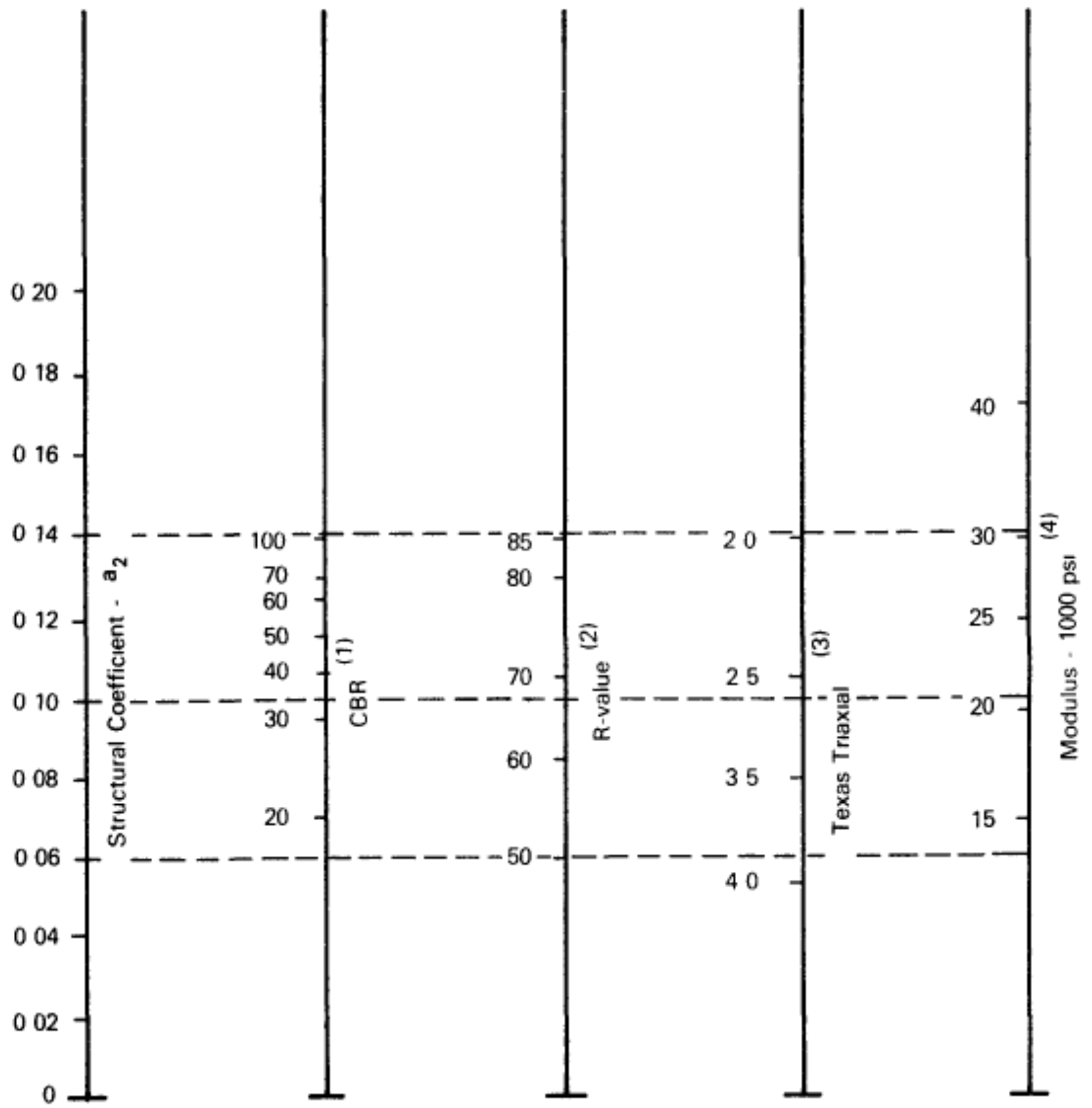
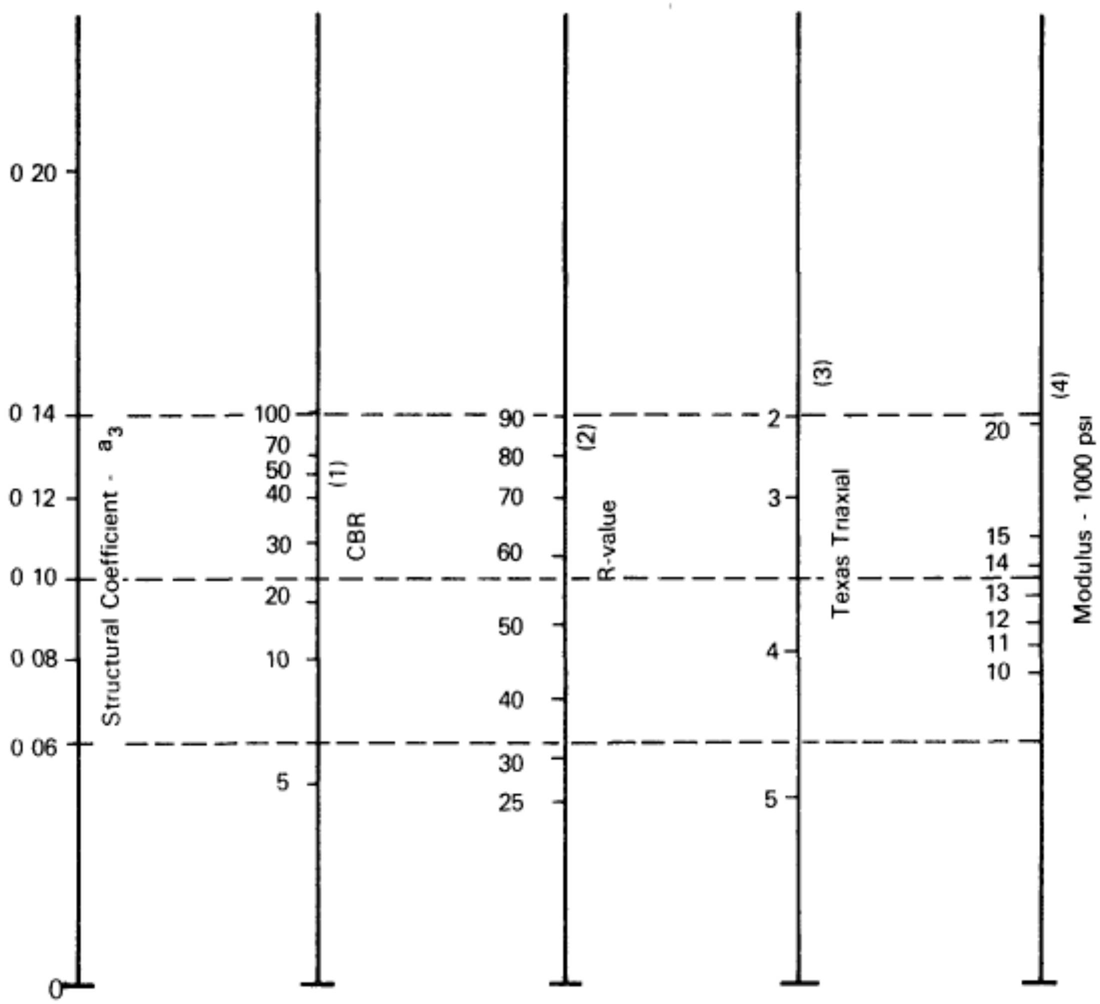


Figure 5.8 *Chart for Estimating Structural Layer Coefficient of Dense-Graded Asphalt Concrete Based on Elastic (Resilient) Modulus (AASHTO, 1993)*



(1) Scale derived by averaging correlations obtained from Illinois
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming
(3) Scale derived by averaging correlations obtained from Texas
(4) Scale derived on NCHRP project *(3)*

Figure 5.9 Variation on Granular Base Layer Coefficient (a2) with Various Base Strength Parameters (AASHTO, 1993)



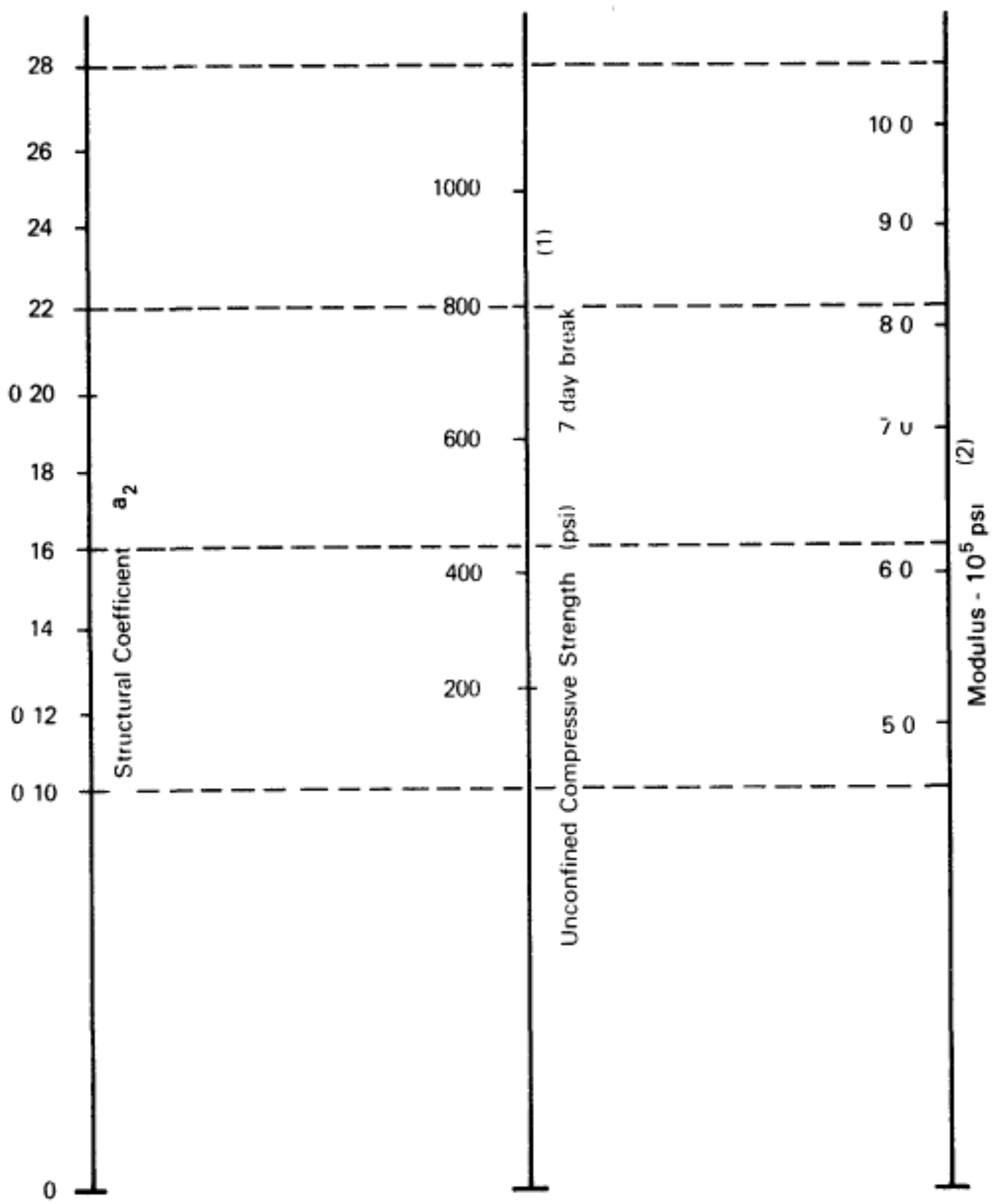
(1) Scale derived from correlations from Illinois

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming

(3) Scale derived from correlations obtained from Texas

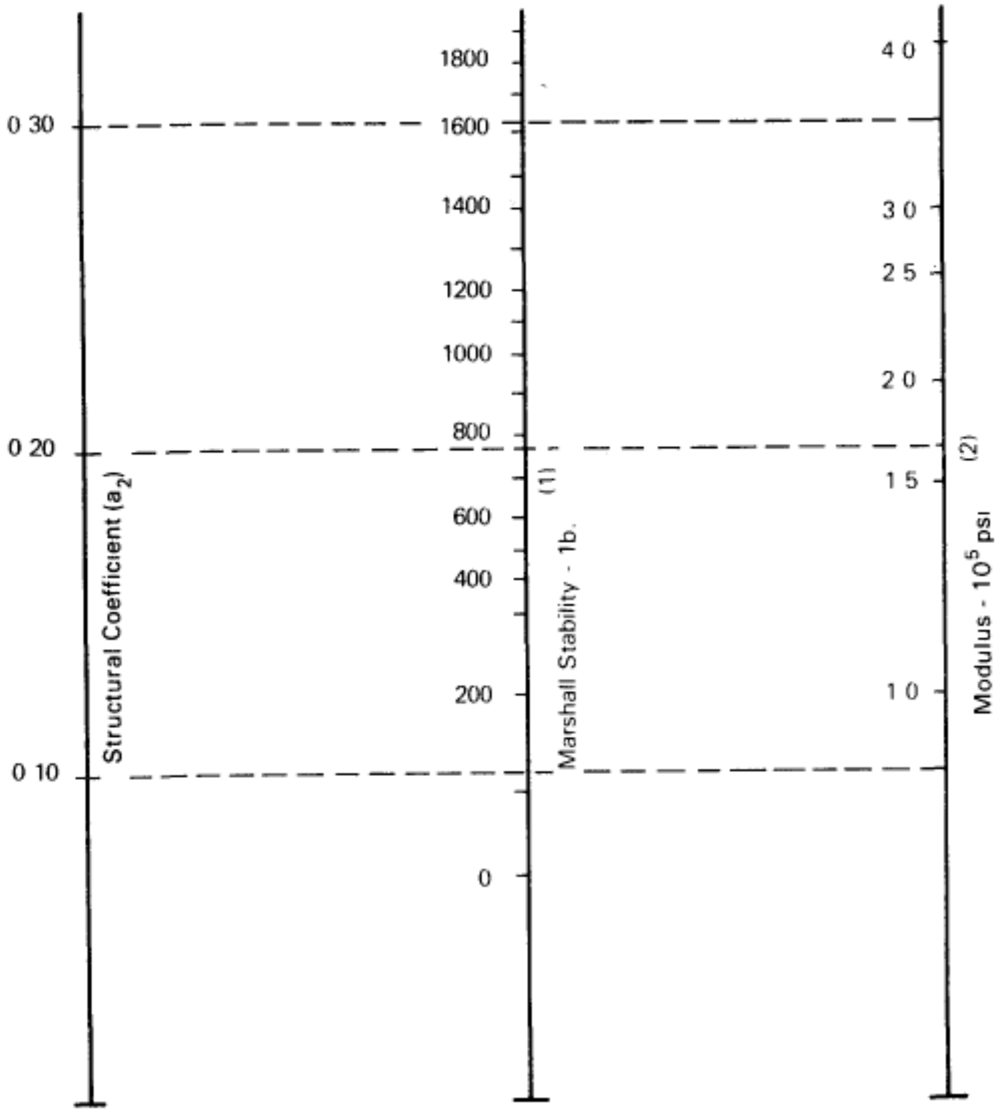
(4) Scale derived on NCHRP project *(3)*

Figure 5.10 Variation in Granular Subbase Layer Coefficient (a3) with Various Subbase Strength Parameters (AASHTO, 1993)



(1) Scale derived by averaging correlations from Illinois Louisiana and Texas
(2) Scale derived on NCHRP project (3)

Figure 5.11 Variation in a for Cement-Treated Bases with Base Strength Parameter (AASHTO, 1993)



(1) Scale derived by correlation obtained from Illinois

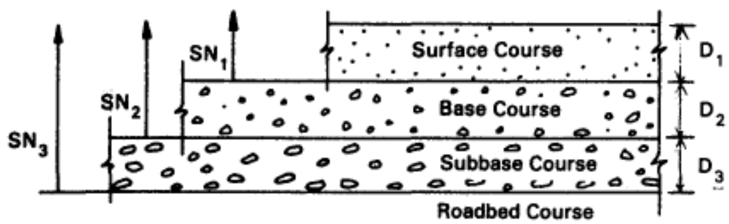
(2) Scale derived on NCHRP project (3)

Figure 5.12 Variation in a2 for Bituminous-Treated Bases with Base Strength Parameter (AASHTO, 1993)

Table 5.7 Recommended m_i values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements (AASHTO, 1993)

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	**Less Than 1%**	**1–5%**	**5–25%**	**Greater Than 25%**
Excellent	1 40–1 35	1 35–1 30	1 30–1 20	1 20
Good	1 35–1 25	1 25–1 15	1 15–1 00	1 00
Fair	1 25–1 15	1 15–1 05	1 00–0 80	0 80
Poor	1 15–1 05	1 05–0 80	0 80–0 60	0 60
Very poor	1 05–0 95	0 95–0 75	0 75–0 40	0 40

When designing a pavement using AASHTO 93, the nomograph in Figure 5.7 is used in an iterative process. The thickness of each layer above the subgrade is determined by using the resilient modulus of the underlying layer on Figure 5.7. A "top-down" approach is then used to determine the required thickness of each layer based on the stiffness of the layer beneath it.



$$D^*_1 \geq \frac{SN_1}{a_1}$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN^*_1}{a_2 m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (SN^*_1 + SN^*_2)}{a_3 m_3}$$

1) a, D, m and SN are as defined in the text and are minimum required values

2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value

Figure 5.13 Determining Thicknesses of Layers Using a Layered Analysis Approach

Table 5.8 gives the minimum thicknesses of the asphalt concrete and the aggregate base layers for different levels of traffic loading (ESAL).

Table 5.8 Recommended Minimum Layer Thicknesses (inches)

Traffic, ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1 0 (or surface treatment)	4
50,001–150,000	2 0	4
150,001–500,000	2 5	4
500,001–2,000,000	3 0	6
2,000,001–7,000,000	3 5	6
Greater than 7,000,000	4 0	6

Example:

An interstate highway pavement composed of an HMA surface course, a crushed-stone granular base course, and a sand/gravel subbase course is to be designed for a design period of 20 years, and a cumulative load of 1.2 x 10^6 ESALs. The quality of drainage is considered fair because water can be removed from the pavement structure within one week. However, there is a large amount of precipitation, so more than 25% of the time the pavement will be exposed to moisture levels approaching saturation. The material properties are as follows:

- Effective roadbed soil resilient modulus = 5,500 psi
- Resilient modulus of subbase course = 15,000 psi
- California Bearing Ratio of the base course = 100
- Resilient modulus of HMA = 4.3 x 10^5 psi

Determine the thicknesses of the surface, base, and subbase courses required using the AASHTO 93 method.

AASHTO 93 is generally no longer used for asphalt pavement design, however the design considerations within the method are still just as relevant to pavement design. As such, familiarity with AASHTO 93 provides good insight into flexible pavement design.

5.4.3. Mechanistic-Empirical Pavement Design Guide (MEPDG)

The development of the next generation of the AASHTO *Design Guide* began in 1998 under the National Cooperative Highway Research Program (NCHRP) Project 1-37A [NCHRP 2004, Tighe 2007, Ali 2005]. The objective was to develop a mechanistic-empirical pavement design guide that would address the limitations of AASHTO 93. The MEPDG incorporates major upgrades in pavement and materials technology and the latest mechanistic-empirical design procedures.

The MEPDG includes the procedures for designing both flexible and rigid pavements; however, in this chapter only the flexible pavement design procedure is described. The MEPDG uses the calibrated design procedure that allows the integration of material characterization, climate conditions and traffic loading in pavement design.

Although the rationale for a mechanistic-empirical design method has a sound base of science and engineering, it should not be adopted too quickly, and care is required during implementation. Most road agencies have built up proven design practices over many years and will want to preserve that experience. Mechanistic-empirical methods will require model calibration and validation. The three stages of pavement design in the MEPDG are shown in Figure 5.14.

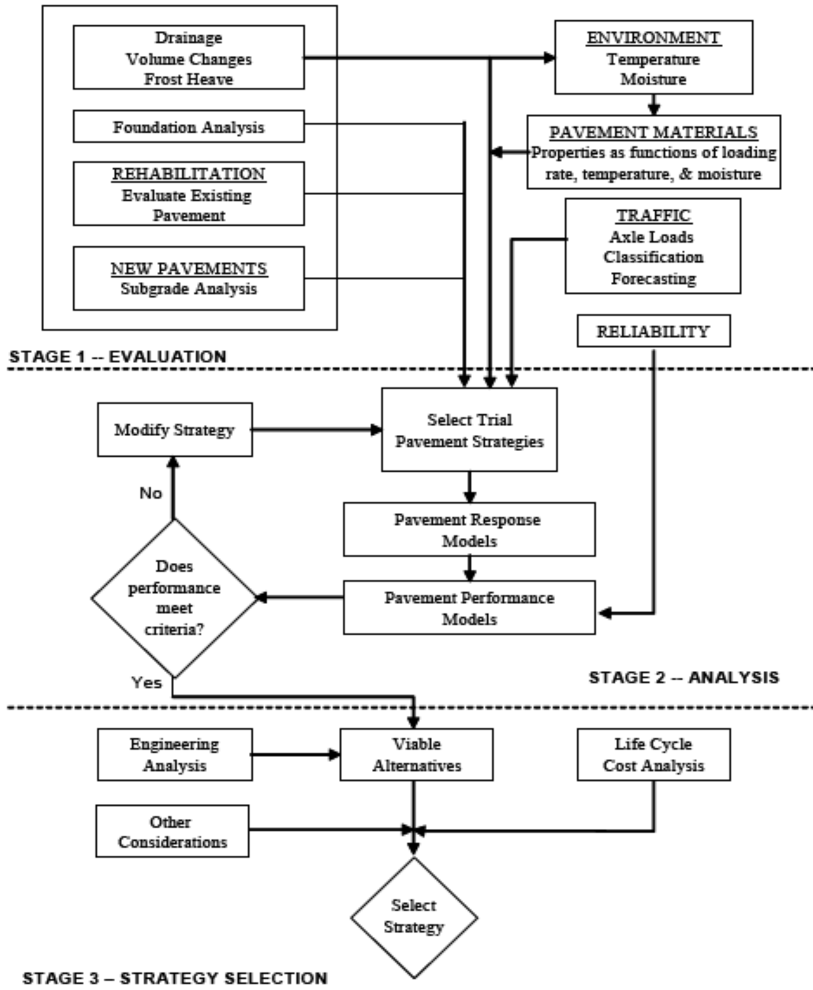


Figure 5.14 Three-stage pavement design process in MEPDG [NCHRP 2004]

The MEPDG uses a hierarchical approach for determining the design inputs. Three levels of input can be selected depending on the importance of the project and the availability of data.

- **Level 1** – is directed to the highest accuracy and reliability. It is used on the most heavily trafficked roads or where safety and economic consequences of early pavement failure are severe. It requires direct laboratory or field testing, such as dynamic modulus of hot mix asphalt, non-destructive deflection testing for the rehabilitation of existing pavements, and site-specific axle load spectra.

- **Level 2** – is consistent with much of the current practice. The inputs are based on limited testing and/or are selected from the agency database and can be estimated empirically. Examples include determination of the dynamic modulus from the grade of asphalt cement, aggregate and mix properties or site-specific traffic volume and classification used with the agency specific axle load spectra.

- **Level 3** – is the lowest level of MEPDG design. It can be used on low volume roads where the consequences of early failure are not severe. Inputs would be user selected default values such as dynamic modulus for a given mix type or default axle load spectra for a certain class of road.

The following main eight items are considered in the MEPDG:

1. Pavement performance
2. Traffic
3. Subgrade soil
4. Material characterization
5. Distress prediction modeling
6. Environment
7. Drainage
8. Reliability

MEPDG provides several advantages over AASHTO 1993:

- It is calibrated based on a much larger data set that is more representative of conditions and higher traffic levels (calibrated using LTPP pavement sections, etc.).
- It provides more accurate performance predictions, and in turn produces more cost-effective designs.
- It provides greater flexibility in evaluating the effects of various pavement materials, traffic loading conditions, design features, and construction practices.
- The procedure can be used to evaluate failures and assess the factors contributing to good performance.
- The distress prediction models can be calibrated for local conditions and updated to reflect new materials and performance data.

As noted above, one of the more significant advantages of MEPDG is the ability to calibrate the design equations so that they more accurately represent local conditions. MEPDG has been calibrated to data collected across North America; however, local materials and practices may cause differences in performance. The MEPDG model coefficients can be adjusted to provide better predictions of pavement distress for local climate, materials, and construction practices.

5.4.3.1. Pavement Performance

Pavement performance is characterized in the MEPDG by roughness as indicated by the International Roughness Index (IRI). Initial and terminal IRI values are established and the roughness model is used to assess whether the pavement section can achieve the design life. The roughness of the pavement depends on the initial roughness achieved after construction, pavement distresses such as cracking and rutting, subgrade condition and age, and pavement maintenance activities. The roughness model used in MEPDG is defined as:

Equation 5-7

where:

$S(t)$ = pavement smoothness at a specific time, t (IRI, m/km)

S_0 = initial smoothness immediately after construction (IRI, m/km)

$S_{D(t)(i)}$ = change of smoothness due to i^{th} distress at a given time t in the analysis period

a_i, b_j, c_j = regression constants

S_j = change in smoothness due to site factors (subgrade and age)

M_j = change in smoothness due to maintenance activities

This approach requires extensive calibration and validation [NCHRP 2006].

5.4.3.2. Traffic Characterization

Unlike AASHTO 93, where mixed traffic is converted to ESAL, in the MEPDG, traffic is considered in terms of axle load spectra. The full spectra for single, tandem, tridem and quad axles are considered.

5.4.3.3. Subgrade Soil

The MEPDG describes subgrade characterization, subsurface exploration, laboratory testing and subgrade improvements and strengthening. The main characterization of the subgrade soil is by resilient modulus (M_R) determined by laboratory testing or back-calculated from non-destructive deflection testing. However, agency experience can also be used to supplement these methods for lower input levels; this may include the estimate of M_R based on soil classification and moisture condition or determining CBR and transforming it to M_R using correlation equations.

5.4.3.4. Pavement Materials Characterization

Two categories of material properties are considered: response properties; and distress properties. The response properties such as elastic modulus and Poisson's ratio are required to calculate the states of stress, strain and displacement within the pavement structure. The distress properties are used through transfer functions to predict the major mode of pavement distress, such as cracking and rutting.

The form of the elastic modulus used to characterize asphalt mixes is the dynamic modulus that varies with temperature, rate of loading and age of mix. In the MEPDG the relationship between the dynamic modulus, mix temperature and the time rate of loading is referred to as the master curve. The master curve, coupled with an asphalt binder aging model, can be used to determine the mix modulus at incremental points in the design analysis period. Depending on the level of input, the dynamic modulus of asphalt mixes can be measured in the laboratory, determined from the asphalt grade, aggregate characteristics and mix volumetrics, or default values based on the mix type can be used for the lowest input level.

A nonlinear model is used in the MEPDG to characterize the resilient modulus of unbound materials including granular base and subbase and also for characterizing the subgrade soils. In Level 1, the resilient modulus of the unbound materials should be measured and nonlinear coefficients. In Levels 2 and 3, the modulus can be correlated using predictive equations to CBR, R values, plasticity/gradation properties or to basic soil classification test result. For pavement rehabilitation, it is recommended that the M_R is determined using FWD measurements.

5.4.3.5. Distress Prediction Models

The distress prediction models of flexible pavements in the MEPDG is related to the pavement structural response in terms of stresses, strains and displacements at critical locations in the pavement structure due to traffic loading and environmental factors. The structural distresses considered in the design are:

- Bottom-up fatigue cracking
- Top-down fatigue cracking (longitudinal)
- Thermal cracking
- Permanent deformation (rutting)
- Reflection cracking

In the MEPDG analysis, asphalt is assumed to be linear elastic. A multi-layer elastic program JULEA is used to calculate the structural response in the asphalt layer. The estimate of the fatigue damage is based on Miner's law:

Equation 5-8

where:
D = damage
r = total number of periods
n_i = actual traffic for period *i*
N_i = traffic allowed under conditions prevailing in *i*

The fatigue model used in the MEPDG to determine the allowable number of load repetitions to onset of cracking is:

$$N_f = 0.00432C(1/\varepsilon_t)^{3.291}\,(1/E)^{0.854}$$

Equation 5-9

C	=	10^M
M	=	$4.84\{[V_b/(V_a + V_b)]-0.69\}$

where:

N_f	=	number of cycles to failure
C,M	=	constants
ε_t	=	tensile strain at the bottom of asphalt (mm/mm)
V_b	=	effective binder content by volume (%)
V_a	=	air voids (%)
E	=	asphalt modulus

The MEPDG has abandoned the traditional vertical subgrade strain criterion for rutting. It recognizes that the unbound materials are non-linear and exhibit stress dependent properties. In order to address the non-linearity in the load response of the unbound materials, a finite element program is used in the MEPDG to determine the response of these layers. The total surface pavement rutting is estimated in the MEPDG as the sum of rutting in the asphalt layers, granular layers and subgrade.

$$\boldsymbol{RD} = \sum_{\boldsymbol{i=1}}^{\boldsymbol{nsublayers}} \boldsymbol{\varepsilon_p^i\, h_i}$$

Equation 5-10

where:

RD	= rut depth (mm)
nsublayers	= number of sublayers
ε^i_p	= total plastic strain in sublayer i (mm/mm)
h^i	= thickness of sublayer i (mm)

Rutting in the asphalt layer is determined using the following model:

$$\varepsilon_p / \varepsilon_r = a_1 T^{a2} N^{a3}$$

Equation 5-11

where:

ε_p	= accumulated plastic strain at N repetitions of load (mm/mm)
ε_r	= resilient strain of the asphalt material as a function of the mix
N	= number of load repetitions
T	= temperature (deg C)

a_i = non-linear regression coefficients

The numerical coefficients are obtained from laboratory repeated load tests on asphalt mixtures that require field verification to determine the final distress model. Field calibration factors, β_{ri}, are then added to the equation:

$$\varepsilon_p / \varepsilon_r = \beta_{r1}\, a_1 T^{a2} N^{a3\beta r2}$$

Equation 5-12

5.4.3.6. Environment

The two main environmental factors included in the MEPDG are temperature and moisture. Both may have a significant impact on pavement performance. Their changes can impact pavement material properties, and hence strength, durability and load carrying capacity. The MEPDG incorporates the Enhanced Integrated Climatic Model (EICM) to simulate changes in the behaviour and characteristics of pavement materials related to changes in temperature and moisture conditions. The EICM has three components: climatic material structural model, frost heave and thaw settlement model; and infiltration and drainage model.

The climatic module in the MEPDG requires the designer to select a climate data file for the location closest to the project being designed. Multiple climate stations are included in DARWin-ME and can be used to develop a virtual climate model for the particular design.

5.4.3.7. Drainage

In AASHTO 93, the impact of drainage conditions was addressed through the provision of drainage coefficients (m_i). The coefficients were a function of climatic factors and the drainability of the base materials. In the MEPDG, the impact of providing subsurface drainage is addressed in the adjustments to the distress models. The following drainage and surface parameters are considered: shortwave absorptivity, infiltration, drainage path length, and pavement cross-slope. Infiltration input is not required for flexible pavements. The EICM model predicts how drainage conditions impact the modulus of unbound granular layers and subgrade. In the MEPDG the need for a subdrainage system is also analyzed. It includes the following steps: assessing the need for drainage based on climate, subgrade and pavement materials conditions; selection of drainage alternatives; hydraulic analysis; preparing pavement cross sections; and performing structural analysis. The MEPDG also includes recommendations for the assessment of drainage needs, hydraulic design of drainage systems, material requirements for drainable layers and construction and maintenance issues.

5.4.3.8. Reliability

In the MEPDG, reliability is defined as a probability that the development of each of the key distresses will not cause the required smoothness levels to fall below a selected critical level over the design period. Design reliability is defined as the probability that each of the key distress types and roughness will be less than a selected critical level over the design period. It can be run in both a deterministic and probabilistic manner.

R = P [IRI or Distress over Design Period < Critical IRI or Distress Level]

For roughness , the design reliability is defined as follows:
R = P [IRI over Design Period < Critical IRI Level]

As with all pavement designs, the reliability of the MEPDG is dependent on the ability to predict and model a wide range of factors that introduce a substantial degree of variability and uncertainty. These factors include traffic volumes and axle distributions, material properties, construction quality, and pavement performance modeling errors and calibration errors. There are two methods to address these uncertainties: deterministic and probabilistic. In the deterministic method each design factor is represented by a value selected by the designer. In the probabilistic method each design factor is represented by a mean and a variance.

5.5 Rigid Pavement Design

A rigid pavement generally consists of plain, composite or reinforced concrete slabs placed on a base or subbase, or sometimes directly on subgrade. Concrete is a somewhat unique paving material in that it has a high modulus of elasticity, which makes it very rigid. The rigidity and beam strength of a concrete pavement allows it to distribute loads over large areas. This has led to a set of design requirements and procedures, which are described in the following sections. It is important to note that there are many design variables which need to be optimized for a successful design. Thickness is only one such variable; jointing and base support are equally important issues to address.

5.5.1. Design Types

As mentioned previously and outlined, there are several concrete pavement types, including the following:

1. Plain concrete pavements with either dowelled or undowelled joints, usually referred to as jointed plain concrete pavements (JPCP).
2. Jointed reinforced concrete pavements (JRCP), typically with dowelled joints.
3. Continuously reinforced concrete pavements (CRCP), constructed without joints.
4. Prestressed or post-tensioned concrete pavements.

The most common type in Canada and other jurisdictions is the JPCP. This pavement structure is built without reinforcing steel and may or may not include dowels in the transverse joints. Typically, a maximum joint spacing of 4.5 m is used to minimize transverse slab cracking.

Another type of concrete pavement gaining acceptance in Canada is the concrete overlay. These pavement types involve placing a new, relatively thin layer of concrete on top of existing concrete, asphalt, or composite pavements. The overlays add structural capacity and can be designed as either a bonded or unbonded type.

Bonded overlays can be placed on pavements that are in relatively good condition and serve to increase the structural capacity, by engaging the existing pavement through the bond. In these pavements, flexural loads are distributed throughout the full depth of the section, resulting in relatively low tensile stresses at the bottom face of the pavement. The existing pavement must be in good condition, or repaired prior to placing the overlay, or else cracks will reflect up through the new overlay, drastically reducing the durability. The joints in bonded overlays are aligned with the joints in the existing pavement.

Unbonded overlays can be placed on pavements in poor to good condition. These overlays include a separation layer between the new overlay and the existing pavement that serves as a stress reliever. The separation layer usually consists of 25 mm of high-stability HMA or a geotextile sheet, and results in two separate pavement layers. The existing pavement serves as a rigid base in these cases, and provides vertical support, but is not engaged with the surface layer in flexure. The joints in unbonded overlays should be spaced at no more than 18 times the thickness of the overlay, to prevent shrinkage, temperature, and loading cracks.

Reinforced pavements usually contain a steel mesh, which is designed to hold the one or two cracks which usually develop between the joints together. Joint spacings in this design are typically increased to not more than 13.0 m. Due to the longer slab lengths, there is increased joint opening in cold seasons necessitating the use of dowels at joints to provide load transfer. Generally, reinforced pavements have lost favour in northern climates. JRCP usually has a higher capital cost along with potential problems of reinforcement corrosion and joint contamination with incompressible materials which, in turn, may lead to "*blow-ups*".

Continuously reinforced pavements, as the name implies, contain relatively heavy, continuous steel reinforcement in the longitudinal direction to eliminate joints. The steel is designed to ensure that transverse cracking develops at close intervals. The reinforcement also holds the cracks together tightly to maintain a high degree of load transfer. The initial cost for this pavement is significantly higher than for a plain pavement; however, they have been used by various agencies in North America and Europe on high volume facilities, usually in more temperate climatic zones like the Southern U.S.

Prestressed or post tensioned pavements have been used in specialized locations, predominantly airports, where the tensioned steel can be utilized to reduce the concrete thickness. It is more difficult to take advantage of this in a highway pavement and as a result, usage to date has generally been experimental in nature.

Concrete used in pavement has a modulus of elasticity ranging from 25 to 35 GPa and thus has a high degree of rigidity. Concrete pavements also have substantial beam strength as indicated by flexural strengths generally averaging above 4.4 MPa. This rigidity and beam strength enable concrete pavements to distribute applied loads over a large area, which means the pressures exerted on the underlying foundation are a fraction of the applied load.

Bending in a concrete pavement due to a wheel load produces both compressive and flexural stresses. The concrete's high modulus of elasticity often means it has very high compressive strength and relatively low flexural strength. As the compressive stresses induced in a concrete pavement are low as compared to the concrete compressive strength, these stresses are not significant in thickness design. However, tensile stresses in a concrete pavement can be high when compared to the flexural strength of the concrete [CPCA 1999].

Concrete flexural strength governs the amount of stress that the concrete can withstand before fracturing. The allowable number of repetitions for a given axle load is determined based on the stress ratio (induced stress divided by the modulus of rupture). For example, a pavement subjected to a flexural stress of 3.5 MPa with a modulus rupture of 4.8 MPa has a stress ratio of 3.5/4.8 = 0.73. Flexural fatigue research on concrete has shown that as the stress ratio decreases, the number of stress repetitions to failure increases. If the stress caused by the applied load is less than 50% of the modulus of rupture of the concrete, the slab may be subjected to an infinite number of load repetitions at that given load level. If the applied load causes stresses greater than 50% of the modulus of rupture then it induces fatigue in the slab. The fatigue criterion is based on Miner's hypothesis: fatigue resistance not consumed by one load repetition is available for repetitions of other loads. Once the accumulated fatigue exceeds 100% there is a potential for fatigue cracks to develop [CPCA 1999].

If the slab thickness is increased, the stress ratio will decrease for the same load application. Similarly, if the concrete strength increases, the stress ratio will also decrease. Hence, strength and thickness properties of the concrete are very important factors to be accounted for in structural design.

The foregoing provides a brief outline of the basic concrete pavement types. Details in the following sections will focus on design of jointed plain concrete pavement (JPCP), as it is the most widely constructed. The basic principles discussed also generally apply to the other design types.

5.5.2. Bases, Subbases and Subgrade

Tests have shown that for a tire with an applied load of 3,000 kg, the pressure beneath a 200 mm slab is in the order of 0.2 MPa or less and radiates out approximately 3.0 m around the load point, as depicted in Figure 5.15.

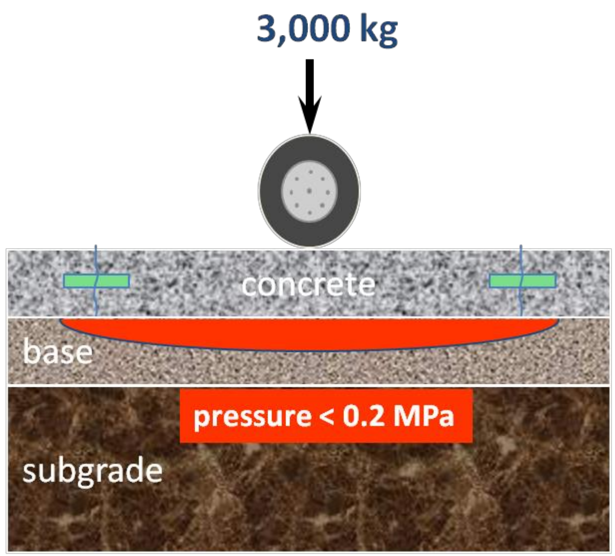


Figure 5.15 Typical load distribution in a concrete pavement [Smith 2001]

Because of *"beam action"*, concrete pavements do not require bases or subbases for structural support. It is, however, very important to provide uniform support. Bases are often used on higher volume roads to facilitate drainage and minimize erosion of foundation materials. They are also often used to expedite construction on lower volume facilities. Non-uniformity of subgrades may lead to differential movement due to frost heave or expansive soils. In concrete pavements extensive removal or sub-excavation of poor subgrade material is generally less economical than control of the problem through subgrade preparation. For example, by using subgrade stabilization in expansive soils or subdrains to eliminate or reduce subgrade moisture levels, frost heave can be significantly reduced. However, if a very deep frost depth condition exists, the reduction of frost heave may only be slight.

A base material is required for facilities which have frequent passage of heavy axle loads and are founded on subgrade soil which is susceptible to erosion. Erosion occurs when heavy vehicles pass over transverse joints causing rapid movement of free water which can remove or relocate fine materials. Redistribution of fine material can lead to a differential in elevation at the joint. This mode of failure, often referred to as faulting, was common prior to the extensive use of dowelled joints and non-erodible base materials.

Faulting and erosion parameters have been incorporated into the latest version of the Portland Cement Association (PCA) and AASHTO thickness design procedures. It is, however, important to remember that water and poor base conditions cannot be corrected by adding to concrete thickness.

Today many high-volume facilities are constructed on permeable layers which significantly reduce the amount of time water is present below the slab. They also eliminate the influence of erodible fine material.

The use of stabilized bases, such as lean concrete bases (LCB) and cement treated bases (CTB), have also been used extensively to provide an erosion resistant base. Many European designs include a jointed LCB. The European practice of jointing the LCB to match overlying joints in the concrete appears to eliminate the random cracking often observed in Canada and the United States.

Additional information on subgrades and subbases for concrete pavements can be found in [ACPA 95].

5.5.3. Joints

Jointing of concrete pavements is required to control the geometry and interval of longitudinal and transverse cracking which would otherwise occur randomly. Joints maintain the long term structural capacity and riding quality of the pavement. Joint design is a very important element in the design of concrete pavements and a well-designed system will:

- Control transverse and longitudinal cracking.
- Accommodate slab movements.
- Provide desired load transfer.
- Provide a reservoir for joint sealant.
- Divide the pavement into practical construction increments.

The development of concrete pavement joint design is based on theoretical studies, laboratory tests, experimental pavements and performance evaluations of existing pavements. Joint design has evolved considerably from a time when no joints were made to the typical short joint spacing used today. Joints are provided to control cracking which results from stresses caused by temperature change, drying shrinkage, moisture differentials, thermal gradients, and traffic loading.

Initial cracking of a concrete pavement is due to a combination of factors. The most significant being shrinkage due to temperature change and loss of water. As concrete hydrates it generates heat but then begins to cool shortly after final set, causing the pavement to contract. After hardening, the pavement is also influenced by temperature and moisture gradients which cause the pavement to curl and warp.

The design of a proper joint system in combination with good construction practices is critical in ensuring the long-term performance of a concrete pavement.

5.5.4. Joint Types

There are four general joint types and they include the following:

1. Transverse contraction joints, which are perpendicular or skewed to the centre line of the pavement.
2. Transverse construction joints, installed at the end of a day's paving or during an extended interruption.
3. Longitudinal joints, typically at centre line or at lane edges.
4. Expansion or isolation joints, placed to isolate the pavement from a more rigid structure or pavement moving on a different axis.

5.5.5. Transverse Contraction Joints

Transverse contraction joints are the dominant type in a concrete pavement and are generally designed to follow the natural cracking pattern. In a JPCP, typical transverse joint spacing should be between 24 to 30 times the pavement thickness. Studies in northern climates have shown that a maximum spacing of 4.5 m will significantly reduce long term transverse slab cracking. It is also important to keep slabs as square as possible with width to length ratios of 1 to 1.25 and a maximum of 1 to 1.5.

Transverse contraction joint design must include a review of load transfer. This is the transfer of load from one side of the joint to the other. In a plain concrete slab load transfer across the joint is provided by the irregular joint face or aggregate interlock (Figure 5.16). This method of load transfer is effective until heavy truck volumes exceed 100 per day per lane. Truck volumes beyond this level will cause aggregate interlock to deteriorate with time.

Where traffic loadings are anticipated to exceed the limits of an aggregate interlock system, smooth, epoxy coated dowel bars are added to provide mechanical load transfer (Figure 5.17). Dowels or load transfer devices also reduce slab deflections and stresses which minimize faulting and reduce corner cracking. Dowel bars are typically spaced on 300 mm centres across the pavement width in North America. In Europe, dowels are often only concentrated in the wheelpaths.

In jointed reinforced concrete pavements (JRCP) dowels should always be used because movement or opening at each joint is much greater than a shorter JCPC. Aggregate interlock is not effective in wide joint openings because the faces of the aggregate no longer bear on the opposite face when a gap is formed.

Transverse joints may be skewed in plain pavements so that each wheel on a vehicle crosses the joint separately. This reduces stresses and deflections of the concrete slab. The potential for faulting is in turn reduced. Some agencies have combined skewed joints with the use of dowels; however, there is no evidence to indicate a skew is beneficial in dowelled joints.

Transverse joints are generally formed by sawcutting the slab to create a reduced cross section. Cracks will then initiate at that reduced cross-section as it will result in a stress concentration. The depth and timing of the sawcut is very important. Historical data has shown that a minimum depth of one-quarter of the slab thickness is necessary to ensure formation of the crack at the joint. Sawcut timing is critical as late sawing will result in the development of random cracks and sawing too soon can cause spalling at the joints. Conventional sawing should begin when ravelling of the sawcut does not occur, typically between 4 and 12 hours after placement of the concrete. Transverse contraction joints should be designed to intersect catchbasins or other structures to ensure random cracks will not propagate from the structure. Additional details may be found in [ACPA 91b and PCA 92a].

Early-entry saw cuts are gaining wider use in the concrete pavement industry. This method is a dry-process which can begin as soon two hours after the pavement surface is finished. The sawcuts in this process are typically no more than 25 mm, and generally result in more distributed cracking than conventional sawcutting (McGovern, 2002).

5.5.5.1. Transverse Construction Joints

Transverse construction joints are used at planned interruptions such as at the end of the construction day, and where unplanned interruptions suspend activities for extended times. Planned construction joints should be located at a transverse contraction joint location and should be dowelled as they are generally formed with a smooth vertical face (Figure 5.18).

5.5.5.2. Longitudinal Joints

Longitudinal joints are used to prevent random longitudinal cracking which is caused by the combined effects of load and restrained slab warping. Typically, longitudinal joint spacing varies from 3.0 to 4.0 m and can be aligned to delineate the traffic or parking lanes in urban areas.

There are two basic types of longitudinal joints; one is used when two or more lanes are constructed at a time and the other is used when one lane is constructed at a time. With one lane at a time construction, a longitudinal joint is formed with a keyway to provide load transfer across the joint (Figure 5.19). When two or more lanes are paved at a time, the longitudinal joint is generally formed by sawcutting the concrete to a depth of one-third the slab thickness (Figure 5.20).

Longitudinal joints may be tied together with deformed steel bars (tiebars) to prevent lane separation. Tiebars are generally used in highway applications where frequent heavy loads can cause the longitudinal joints to separate. On lower volume urban facilities lateral restraint provided by backfilled curbs may negate the need for tiebars. However, on streets with low lateral resistance such as elevated embankments or no curb sections, tiebars should be used.

Typically, tiebars are 15 mm in diameter, 760 mm in length and are spaced at 600 mm intervals. See [ACPA 91b] for additional details.

5.5.5.3. Expansion or Isolation Joints

Isolation joints allow differential horizontal or vertical movements between the concrete pavement and another structure to occur without damage to either. Isolation joints are most common in urban concrete pavements where the pavement must be isolated from manholes or catchbasins or another pavement in a “T” intersection. Generally, the joint is formed a minimum of 300 mm beyond the exposed frame in a circle or half circle to eliminate stress concentrations at corners of square or rectangular patterns (Figure 5.21).

Isolation joints are formed by placing a compressible joint filler material, 12 to 25 mm thick, for the full slab depth. The filler material should be non-absorbent, non-reactive (Figure 5.22). Isolation joints used to abut an existing pavement or a “T” intersection should be constructed with thickened edges (Figure 5.23). Dowels should not be used as lateral movement must be accommodated. Additional information on jointing urban concrete streets can be found in [PCA 92b].

Expansion joints are constructed similarly to isolation joints; however, they are used to relieve compressive stresses in the pavement and are usually dowelled (Figure 5.24). Historically, these joints were placed at regular intervals of 60 to 150 m to relieve compressive stress. Studies have shown expansion joints are detrimental to long-term pavement performance and should not be used [PCAa 92]. Expansion joints allow adjacent contraction joints to open wider, thereby accelerating joint deterioration. Expansion joints are now only recommended where a bridge structure is abutted, or when concrete is placed at ambient temperatures below 4^{o}C.

5.5.5.4. Joint Sealants

Joint sealants are used to minimize the infiltration of incompressibles and surface water into the joint. Incompressibles, such as gravel, can create point bearing pressures leading to spalling and in extreme cases “*blow-ups*”. Sealant performance is generally better in pavements with short joint spacings as there is reduced stress in the sealant.

There are two basic types of joint sealant available, one being liquid or field moulded and the other being pre-formed or a compression seal. The liquid sealants can be either hot or cold poured. Pre-formed sealants are factory moulded and rely on long-term compression recovery for successful sealing.

One of the keys to successful sealant performance is joint preparation. Joint openings should be thoroughly cleaned to assure good bond. Most specifications now require sandblasting of the joint surfaces when liquid sealants are used. The sealant should be recessed 6 to 8 mm below the concrete surface to prevent damage by snowplows. The size and configuration of joint sealant reservoirs is influenced by the type of sealant being used and manufacturers' recommendations should be followed.

Some practitioners prefer to avoid the use of joint sealants in an effort to reduce costs on the basis that sealants can often be easily compromised or require replacement early in the pavement life cycle. For unsealed joints, the sawcut width is typically specified to be thinner (approximately 3-5 mm) since they do not require material to be placed within them, and this reduces the amount of incompressibles which can fit into the joint. There are several studies which indicate that this procedure can work in some circumstances, but that well-prepared seals are often best.

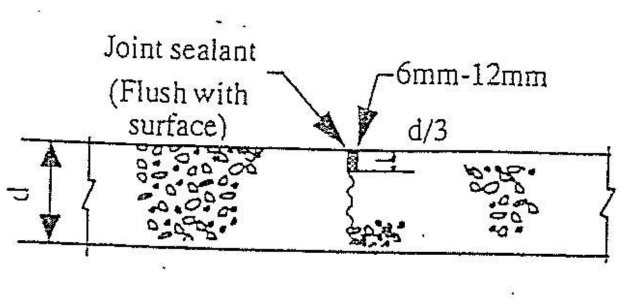


Figure 5.16 Undowelled contraction joint

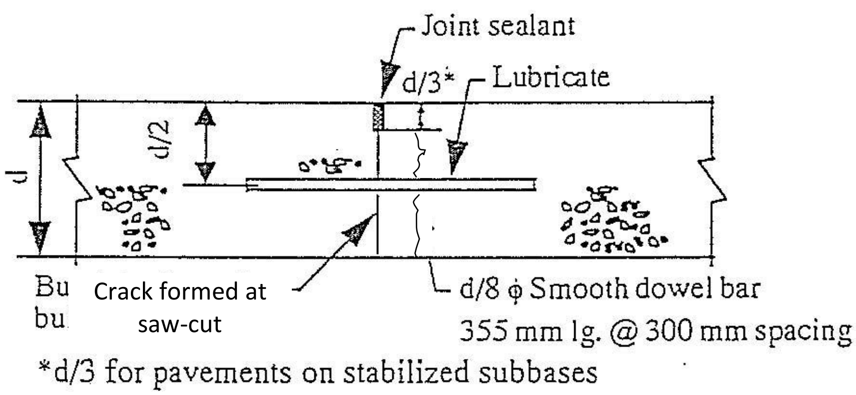


Figure 5.17 Dowelled contraction joint

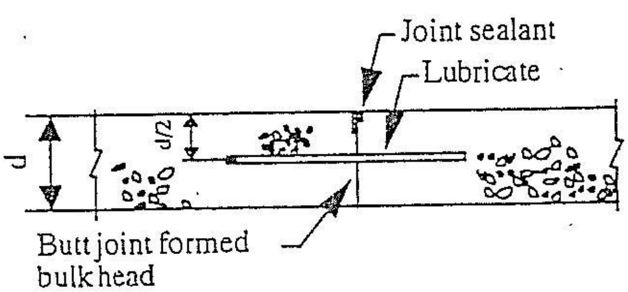


Figure 5.18 Transverse construction joint

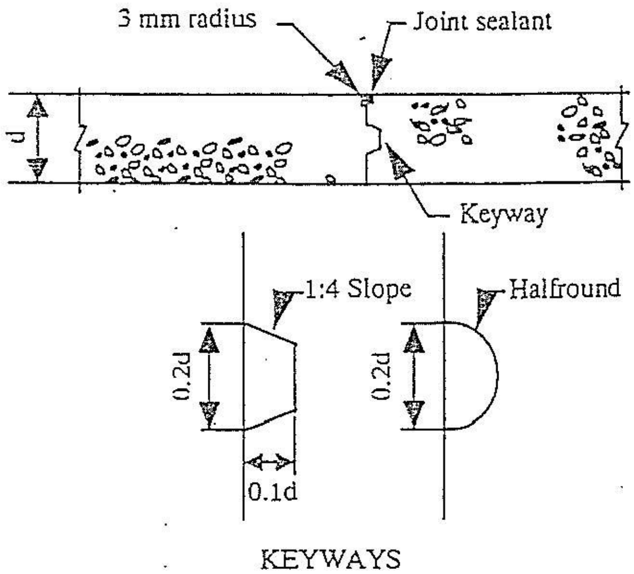


Figure 5.19 Longitudinal keyed joint

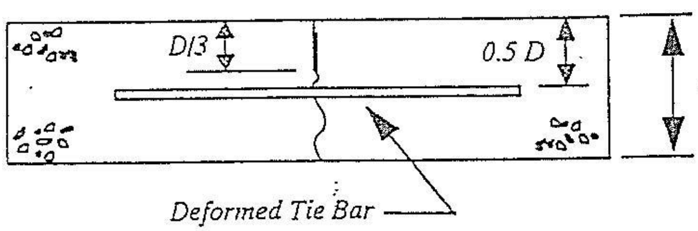


Figure 5.20 Longitudinal sawcut joint

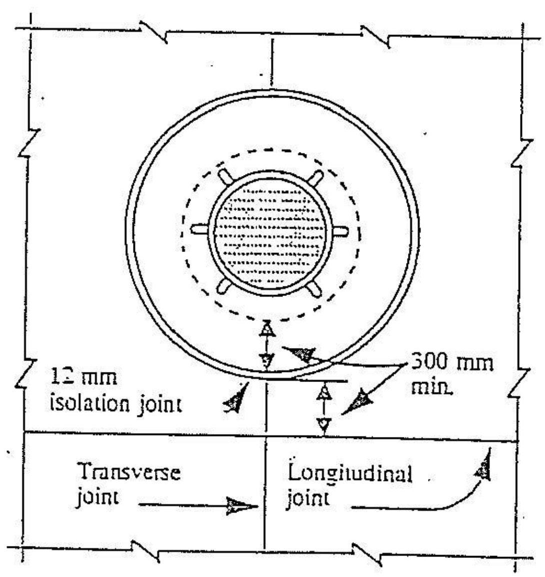


Figure 5.21 Roundouts for appurtenances

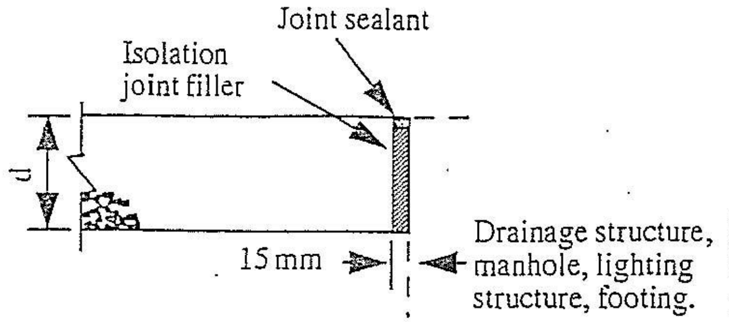


Figure 5.22 Isolation joint

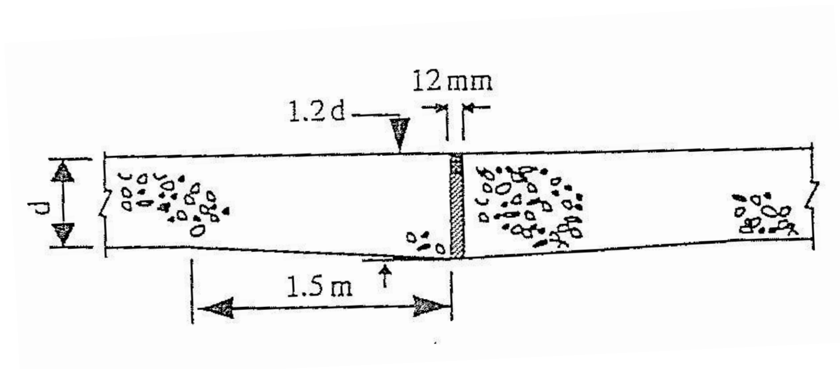


Figure 5.23 Thickened edge isolation joint

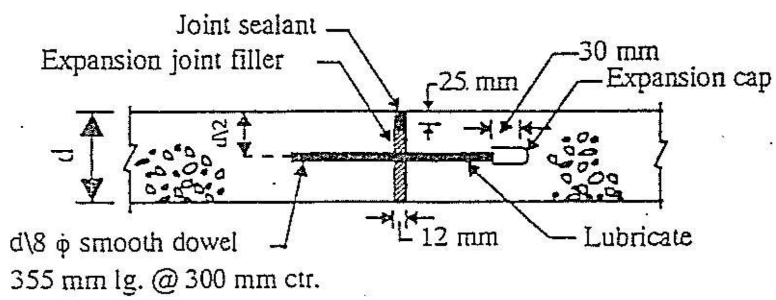


Figure 5.24 Dowelled expansion joint

5.5.6. Thickness Design

Thickness design, along with joint and base design, is an important step in designing a concrete pavement. There are many different approaches to designing the appropriate thickness, ranging from the simple approach of previous experience to the complexities of finite element analysis.

In North America there are two predominant design procedures being utilized. The most popular is the AASHTO design procedure, followed by the PCA design procedure. These are briefly described in the following Section.

5.6 Rigid Pavement Thickness Design

The design procedure recommended by AASHTO is based on empirical equations developed from the results of the AASHO Road Test conducted in Ottawa, Illinois, in the late 1950s and early 1960s.

The basic design equations for rigid pavements are in a similar form to those for flexible pavements and are based on regression analysis of the pavement performance at the AASHO Road Test. These basic equations have since been modified to include variables not considered at the Road Test. The detailed regression equations can be found in [AASHTO 93]. Principal additions or modifications to the original design equations include a provision for variable load transfer at joints, and a drainage coefficient for concrete pavements. Both the flexible and rigid design procedures were modified to incorporate the concept of reliability.

5.6.1. Serviceability

The initial Present Serviceability Index, PSI, (p_o) represents the condition immediately after construction. Using current construction techniques and specifications, high quality concrete roads have initial PSI's of about 4.7 to 4.8. In comparison, the average initial PSI of Portland cement concrete pavements at the Road Test was 4.5. A higher initial PSI will reduce the required pavement thickness as such pavements will outlast a similar pavement built to a lower PSI. This rationale corresponds to the concept of serviceability and justifies specifications that improve initial riding quality. Increasing the value for initial serviceability while holding constant all other input variables in the AASHTO equation demonstrates this relationship.

The terminal PSI (p_t) corresponds to the PSI at which a pavement requires some type of rehabilitation or major maintenance, where p_t varies with the functional class of road. Change in serviceability is also influenced by other parameters such as expansive or frost susceptible soils for which adjustments can be made that are not covered in these notes.

5.6.2. Traffic (ESALs)

Standard weights and axle configurations were used at the Road Test. In practice, there are a variety of axle weights and configurations in a mixed traffic stream. For design, these are converted to Equivalent Single Axle Loads, ESALs.

5.6.3. Design Factors

The performance of rigid pavements is influenced by many design and construction factors. In the AASHTO rigid pavement performance equation, these factors are incorporated as input variables.

5.6.4. Concrete Properties

There are two major properties of concrete which are considered to influence pavement performance in [AASHTO 93]:

S'_c = concrete flexural strength determined at 28-days using third-point loading in a simple beam

E_c = concrete modulus of elasticity

Flexural strength value is based on Canadian Standards Association, CSA A23.2-8C, Flexural Strength of Concrete. The performance equations are based on the average 28-day concrete flexural strength. Strength values measured using other test methods must be converted to the 28-day, third-point method for use in the AASHTO equation.

It is important to use the average strength in the AASHTO procedure, as reliability is used for a safety factor. Most specifications are based on specified strength at the time of construction and it is necessary to correct the specified strength to an average strength. Additional information may be obtained in [ACPA 93].

The other concrete property used in the AASHTO design equation is the modulus of elasticity (E_c). E_c is an indication of how much the concrete will compress under load. In the rigid pavement equation, E_c has only a minor impact on thickness design or projected performance.

5.6.5. Load Transfer Coefficient

One basic item which distinguishes each type of concrete pavement is jointing. Each design provides a different level of load transfer from one side of a pavement joint or crack to the other. The use of steel dowels or continuous reinforcement in the pavement enhances load transfer and the AASHTO rigid pavement design equation includes the J-factor to account for this effect.

Edge support also influences the J-factor. Edge support is provided by widened lanes, tied or integral curb and gutter, or tied concrete shoulders. These improvements generally lead to better performing pavements.

The 1993 AASHTO procedure accounts for edge support in design. A J-factor consistent with the type of pavement and edge support that has been designed is selected.

5.6.6. Subgrade Support

In rigid and flexible pavements, the subgrade eventually carries the load, and pavement performance is affected by the quality of the subgrade. In concrete pavement design, the strength of the soil is characterized by the modulus of subgrade reaction (k).

In many applications, the concrete is not placed directly on the subgrade, but some type of subbase materials is used. When this is done, the k used for design is a “composite k” (k_c) that represents the strength of the subgrade corrected for the additional support provided by the subbase.

By definition, k is determined by the plate load test. The plate load test requires placing a 760 mm diameter rigid plate on the subgrade and applying a load on the plate. The k value is found by dividing the plate pressure by displacement under a known load, expressed as MPa per metre. It can be considered in terms of pressure (MPa) on the subgrade per unit (m) of deflection of the plate. Using the plate load test, the subgrade is modelled as a bed of springs. The value of k is analogous to the spring constant. In fact, k is sometimes referred to as the subgrade “spring constant”. Once k is determined for a given subgrade soil, it is simple to calculate the deflection of the subgrade for any load.

It is important to point out that an error in the value of k has little impact on calculated pavement thickness by the AASHTO rigid pavement equation. For example, an error in the value of k of 100 percent only increases or decreases a typical pavement thickness by about 10 mm.

The 1986 and 1993 versions of the AASHTO Guide [AASHTO 86, 93] provide a loss of support factor. This factor reduces k where a loss of support is expected. A loss of support factor of 1 is equivalent to the conditions at the Road Test.

The use of a loss of support factor is questionable, however, even though it is inherent in the AASHTO design equations. The reason is that loss of support was the primary failure mode of concrete sections at the Road Test. It is also important to remember that a thicker pavement design will not, by itself, solve problems caused by water.

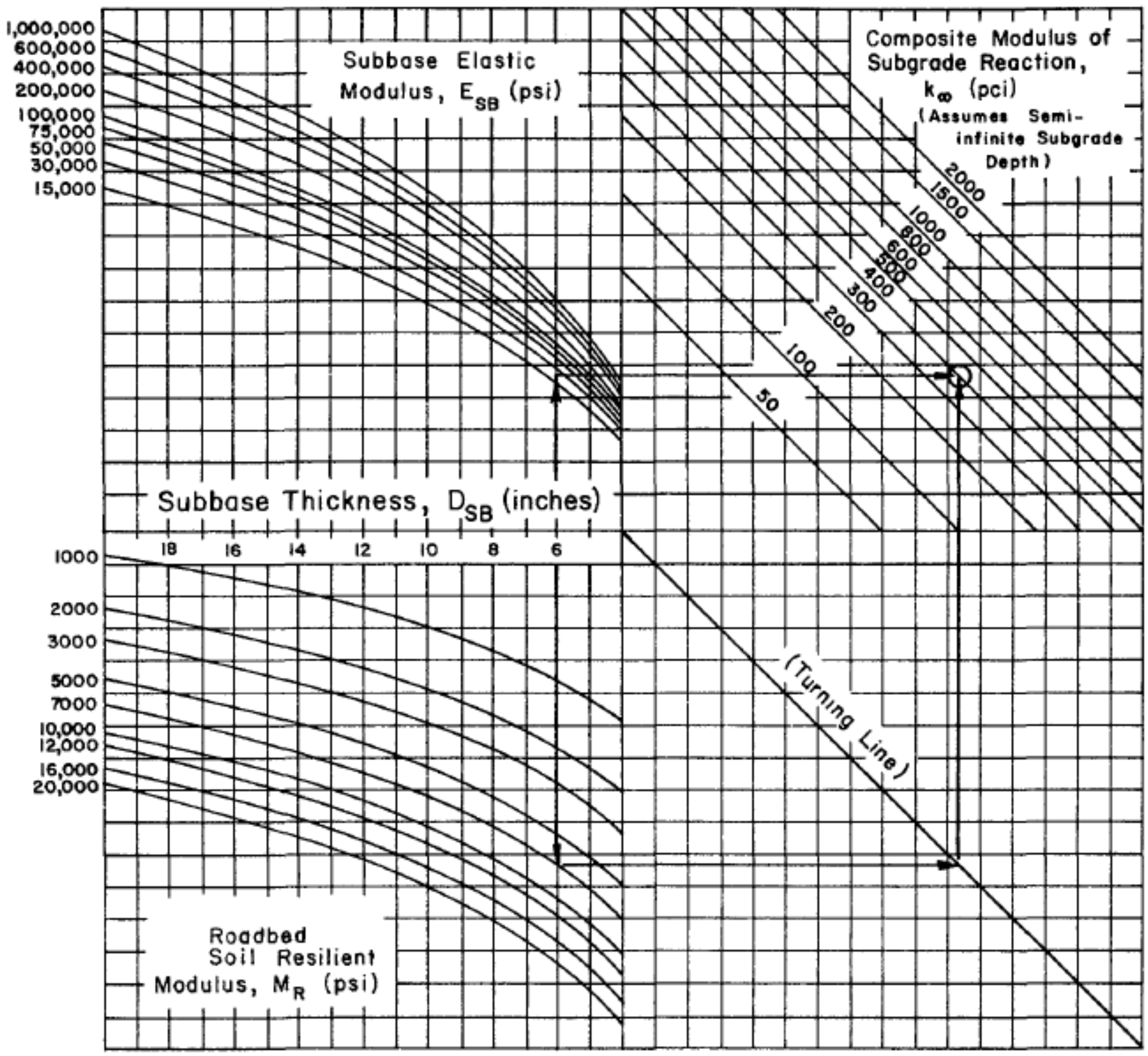


Figure 5.25 Chart for estimating composite modulus of subgrade reaction, k∞, assuming a semi-infinite subgrade depth (AASHTO, 1993)

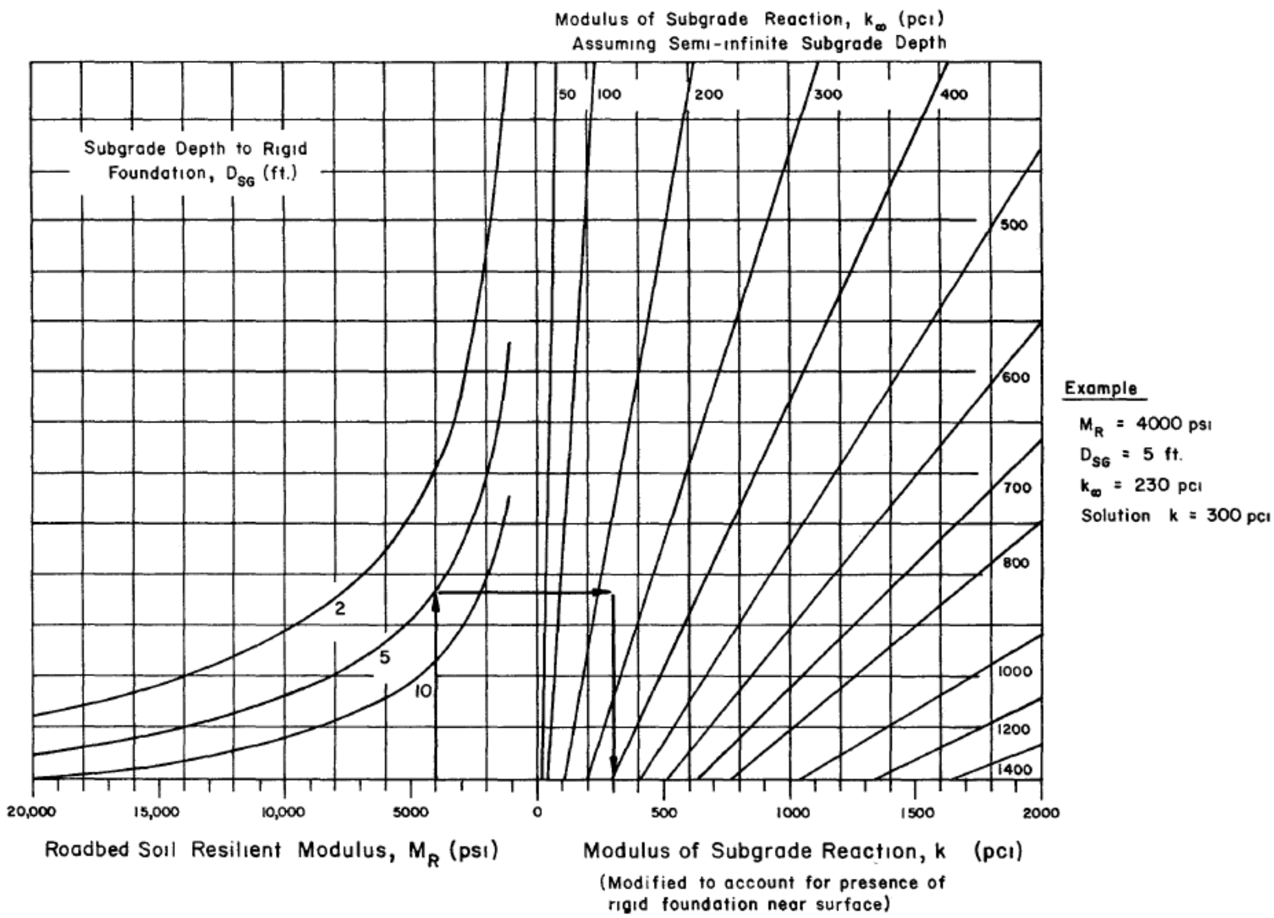


Figure 5.26 Chart to modify modulus of subgrade reaction to consider effects of rigid foundation near surface (within 10 feet) (AASHTO, 1993)

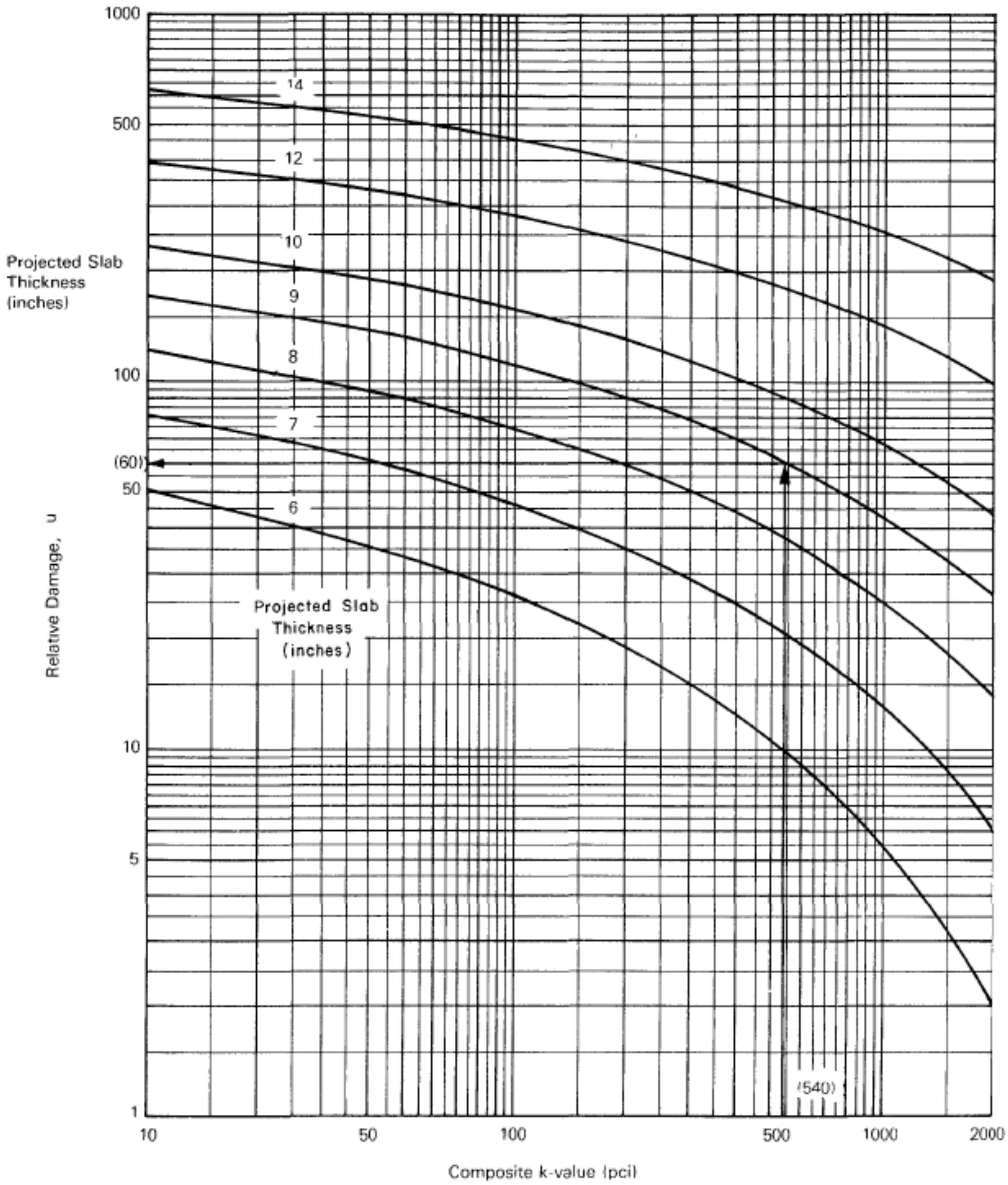


Figure 5.27 Chart for estimating relative damage to rigid pavements based on slab thickness and underlying support (AASHTO, 1993)

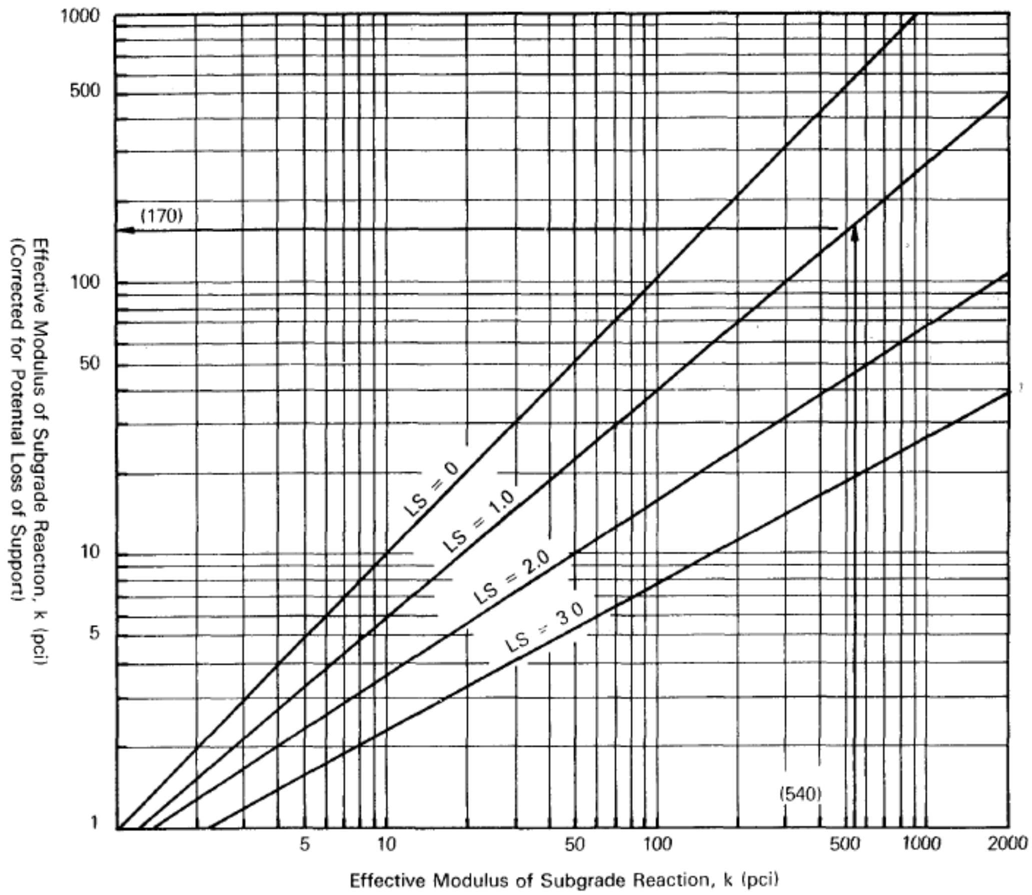


Figure 5.28 Correction of Effective Modulus of Subgrade Reaction for Potential Loss of Subbase Support (AASHTO, 1993)

Table 5.9 Typical Ranges of Loss of Support (LS) Factors for Various Types of Materials (AASHTO, 1993)

Type of Material	Loss of Support (LS)
Cement Treated Granular Base (E = 1,000,000 to 2,000,000 psi)	0 0 to 1 0
Cement Aggregate Mixtures (E = 500,000 to 1,000,000 psi)	0 0 to 1 0
Asphalt Treated Base (E = 350,000 to 1,000,000 psi)	0 0 to 1 0
Bituminous Stabilized Mixtures (E = 40,000 to 300,000 psi)	0 0 to 1 0
Lime Stabilized (E = 20,000 to 70,000 psi)	1 0 to 3 0
Unbound Granular Materials (E = 15,000 to 45,000 psi)	1 0 to 3 0
Fine Grained or Natural Subgrade Materials (E = 3,000 to 40,000 psi)	2 0 to 3 0

NOTE: E in this table refers to the general symbol for elastic or resilient modulus of the material

Example:

A 6” layer of cement-treated granular material ($E=10^6$psi) is to be used as subbase for a rigid pavement. The seasonal values for roadbed resilient modulus and subbase Elastic Modulus are given in the following table. What is the effective modulus of the supporting layers if the bedrock is at a depth of 5 feet? 15 feet? Assume that the concrete slab will be 8 inches thick.

Month	Roadbed Modulus, Mr (psi)	Subbase Modulus, Esb (psi)	Composite k-Value	k-Value on Rigid Foundation	Relative Damage (5 ft)	Relative Damage (15 ft)
January	20,000	50,000				
February	20,000	50,000				
March	3,000	20,000				
April	4,000	20,000				
May	4,000	20,000				
June	8,000	25,000				
July	8,000	25,000				
August	8,000	25,000				
September	8,000	25,000				
October	8,000	25,000				
November	8,000	25,000				
December	20,000	50,000				
				Average		

5.6.7. Coefficient of Drainage

Water is one of the primary contributors to pavement distress. Water can saturate and weaken the subgrade and subbase and pump erodible fines through pavement joints and cracks. This was the primary mode of distress and failure of rigid pavements at the AASHO Road Test.

The 1986 Guide [AASHTO 86] recognized the importance of drainage by incorporating a drainage coefficient (C_d). This coefficient accounts for improved or decreased quality of drainage over those conditions at the Road Test and was carried through subsequent versions of the AASHTO Guide. Table 5.10 provides the coefficients of drainage based on the quality of drainage and the percent of time that the pavement is saturated.

Table 5.10 Coefficient of Drainage (Cd) for Rigid Pavement Support

Quality of Drainage	*Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation*			
	Less Than 1 Percent	*1–5 Percent*	*5–25 Percent*	*Greater Than 25 Percent*
Excellent	1.25–1.20	1.20–1.15	1.15–1.10	1.10
Good	1.20–1.15	1.15–1.10	1.10–1.00	1.00
Fair	1.15–1.10	1.10–1.00	1.00–0.90	0.90
Poor	1.10–1.00	1.00–0.90	0.90–0.80	0.80
Very poor	1.00–0.90	0.90–0.80	0.80–0.70	0.70

5.6.8. Thickness (D)

At the Road Test, slab thicknesses ranged from 62.5 to 312.5 mm (2.5 to 12.5 inches). Therefore, the AASHTO rigid pavement design equation is only valid within this range. If pavement thickness calculations are unusual, it is important to check the design with another procedure such as that developed by the Portland Cement Association.

The AASHTO design procedure may generate a pavement less than 100 mm thick for light traffic streets. A minimum pavement thickness of 100 mm for car traffic and 150 mm for limited truck traffic is generally recommended.

After determining design pavement thickness using estimated values of the input variables or design, it may be advisable to check this against ESAL calculations (note that the term E-18 is used in the AASHTO procedure). In practice, however, this recalculation will probably not significantly affect the new pavement thickness. A summary of the effects of design variables on required thickness and allowable E-18's (ESALs) is given in Table 5.11.**Error! Reference source not found.**

Table 5.11 Effect of changes of design variable on required thickness and allowable ESALs

Design Variable	Effect on

	Required Thickness	Allowable ESALs
Increase Initial Serviceability Index, p_o	Decrease	Increase
Increase Modulus of Rupture, S'_c	Decrease	Increase
Increase Modulus of Elasticity, E_c	Slight Increase	Slight Decrease
Increase Load Transfer Coefficient, J	Increase	Decrease
Increase Coefficient of Drainage, C_d	Decrease	Increase
Increase Modulus of Subgrade Reaction, k	Slight Decrease	Slight Increase
Increase Standard Deviation, S_o	Increase	Decrease
Increase Reliability, R	Increase	Decrease

NOMOGRAPH SOLVES:

$$\log_{10}W_{18} = Z_R{*}S_o + 7.35{*}\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.624{*}10^7}{(D+1)^{8.46}}} + (4.22-0.32p_t){*}\log_{10}\left[\frac{S_c' * C_d\left[D^{0.75}-1.132\right]}{215.63{*}J\left[D^{0.75}-\dfrac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

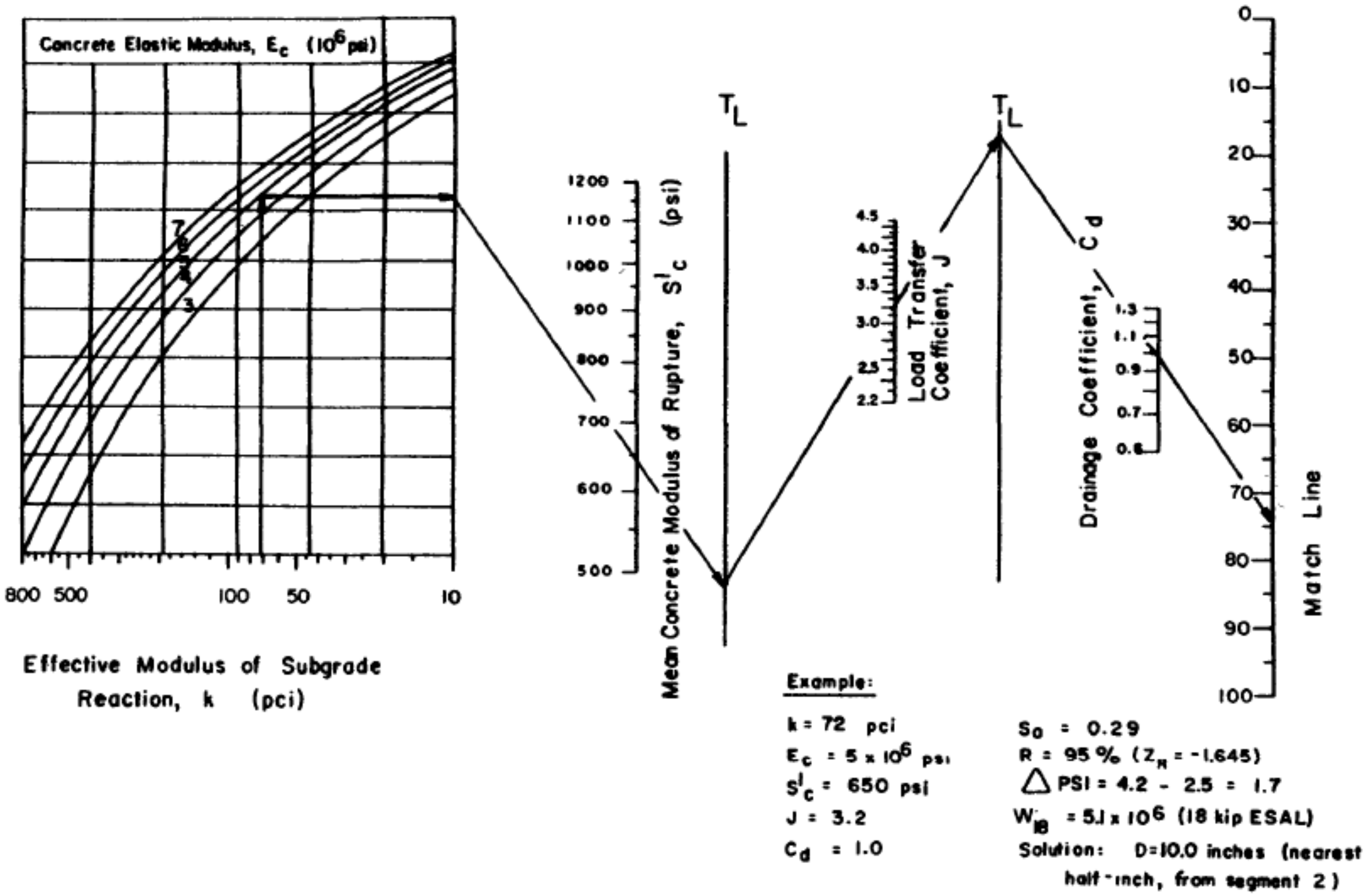


Figure 5.29 Design chart for rigid pavement based on using mean values for each input variable (AASHTO, 1993)

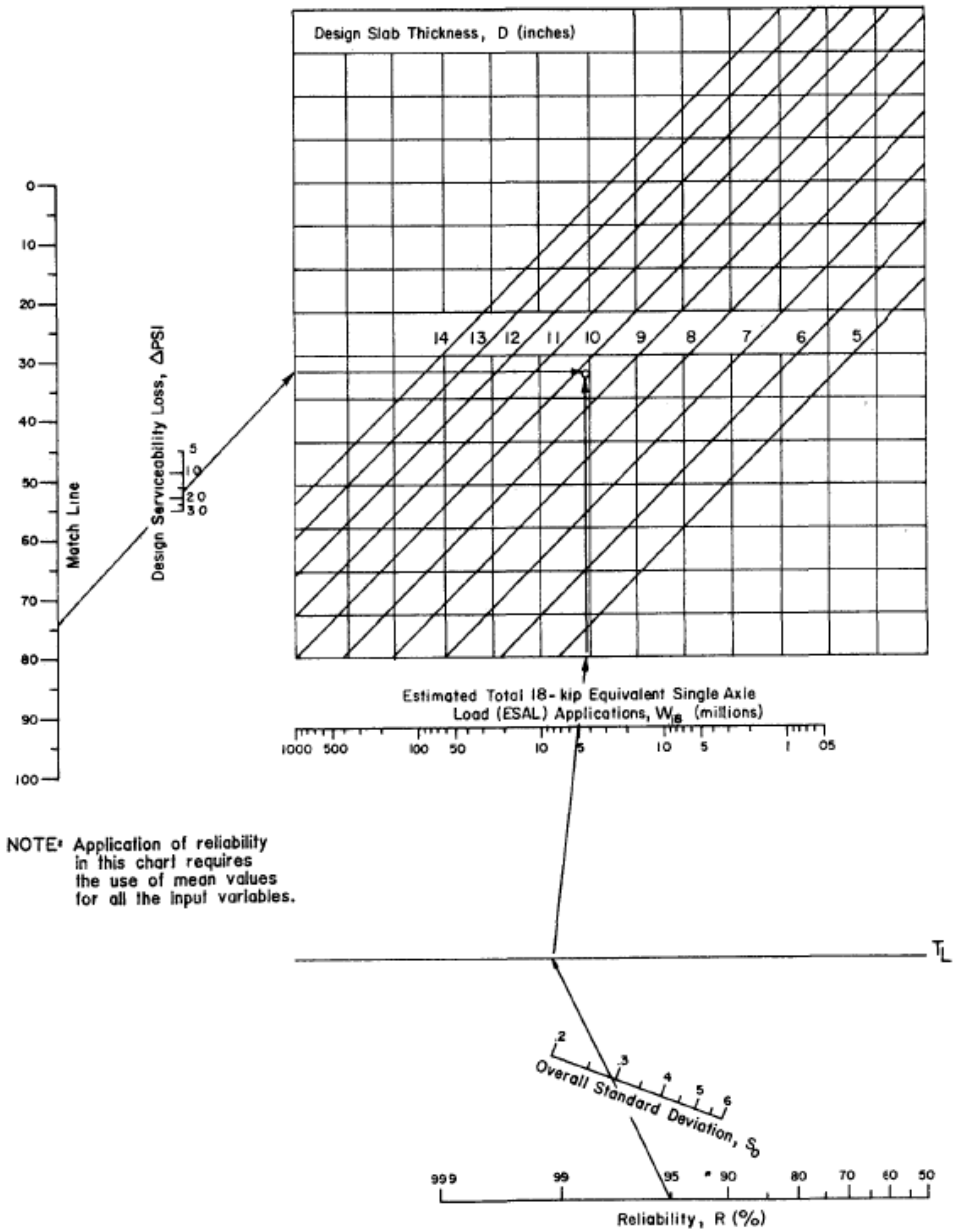


Figure 5.30 *Design chart for rigid pavements based on using mean values for each input variable (pt 2) (AASHTO, 1993)*

Example: Rigid Pavement Design, Required Slab Thickness

Using the corrected modulus values from the previous example, determine the required slab thickness for pavements constructed on each subgrade condition. Use the following design criteria:

- Elastic Modulus of Concrete (E_c)= $5x10^6$ psi
- Modulus of rupture (S_c) of Concrete = 650 psi
- Standard deviation = 3.0, Reliability =95%
- ESAL Application = 2.0 x 10^6 ESALs
- Load Transfer, J = 3.2
- ΔPSI = 4.5 – 2.5 = 2.0
- Pavement has fair drainage and is exposed to saturation 5% of the time

How does this value compare to the assumed thickness in the last example? If not high enough, how should this be accounted for?

References (Chapters 4 through 6):

[AASHTO 86] American Association of State Highway and Transportation Officials, "AASHTO Guide for Design of Pavement Structures", Washington, 1986.

[AASHTO 93] American Association of State Highway and Transportation Officials, "Method of Sampling and Testing", 1993.

[AASHTO 93] American Association of State Highway and Transportation Officials, "AASHTO Guide for Design of Pavement Structures", Washington, D.C., 1993.

[AASHTO 94] AASHTO, "Provisional SHRP Standards, PS-94", Wash., D.C., 1994.

[ACPA 95] American Concrete Pavement Association, "Subgrades & Subbases for Concrete Pavements", TB. 011, 1991.

[ACPA 91b] American Concrete Pavement Association, "Design & Construction of Joints for Concrete Highways", TB-010,1991.

[ACPA 92] American Concrete Pavement Association, "Design of Concrete Pavement for City Streets", Arlington Heights, Illinois, 1992.

[ACPA 93] American Concrete Pavement Association, "Pavement Analysis Software", (PAS), 1993.

[AI 95] Asphalt Institute Superpave Performance Graded Asphalt Binder Specification and Testing Superpave Series No. 1 (SA), Lexington, Kentucky, 1995.

[Anderson 92] Anderson, K.O., J.A. Bervell and S. Teply, "Impact of Changes in Vehicle Weight Legislation on Pavements in Alberta, Canada", Proc., Third Int. Symposium on Heavy Vehicle Weights and Dimensions, Cambridge, U.K., 1992.

[ASTM 94] American Society for Testing and Materials, "Annual Book of ASTM Standards, Volume 04.08 Soil and Rock; Dimension Stone; Geosynthetics", 1994 (Revised Annually).

[CAN/CGSB-163.3-M90] Canadian General Standards Board, "Asphalt Cements for Road Purposes", National Standard of Canada, 1990.

[CGRA 65] Canadian Good Roads Association, "Guide to the Structural Design of Flexible and Rigid Pavements in Canada", Ottawa, 1965.

[CGS 92] Canadian Geotechnical Society, "Canadian Foundation Engineering Manual", 1992.

[CPCA 84] Canadian Portland Cement Association, "Thickness Design for Concrete Highway and Street Pavements", EB209, Toronto, 1984.

[CSA-A23-94] Canadian Standards Association; "Concrete Materials and Methods of Concrete Construction" (A23.1-94) and "Methods of Test for Concrete" (A23.2-94).

[Darter 78] Darter, M.I., "Structural Design for Heavily Trafficked Plain Jointed Concrete Based on Serviceability Performance", Transp. Res. Board, TRR 671, 1978.

[Haddock 93] Haddock, John E., Bo Liljedahl, Anthony J. Kriech and Gerald A. Huber, "Stone Matrix Asphalt: Application of European Design Concepts in North America", Proc., Can. Tech. Asph. Assoc., 1993.

[Kazmierowski 93] Kazmierowski, T.J. and A. Bradbury, "10 Years Experience with Experimental Concrete Pavement Sections in Ontario", Proc., 5'th Int. Conf. on Concrete Pavement Design and Rehabilitation, Purdue Univ. 1993.

[Kennedy 93] Kennedy, T.W., J.S. Mouthrop and G.A. Huber, “Development of SHRP Mixture Specifications and Design and Analysis System”, Proc., Assoc. of Asphalt Paving Tec., Vol. 62, 1993.

[McGovern 02] McGovern, M. (2002, November). *Concrete Technology Today.* Retrieved from Portland Cement Association: http://cement.org/tech/pdfs/ct023saw.pdf

[Mindess 81] Mindess, S., and J.F. Young, “Concrete”, Prentice-Hall Inc., 1981.

[M.M.S. –92] Consulting Engineers of British Columbia, Municipal Engineers Division, “Master Municipal Specifications”, 1992.

[MTO 90] Ministry of Transportation of Ontario, “Pavement Design and Rehabilitation Manual”, SDO-90-01, 1990.

[NCHRP 75] National Co-operative Highway Research Program, Report No. 29, “Treatment of Soft Foundations for Highway Embankments”, 1975.

[OPSS – Rev. 92] “Ontario Provincial Standards for Roads and Municipal Services” (3 Volumes), Ontario Provincial Standards Section, Ministry of Transportation.

[PCA 92a] Portland Cement Association, “Design & Construction of Joints for Concrete Streets”, IA061, 1992.

[PCA 92b] Portland Cement Association, “Proper Use of Isolation and Expansion Joints in Concrete Pavements”, IS400, 1992.

[PCA 92c] Portland Cement Association, “Design of Concrete Pavement for City Streets”, IS184, 1992.

[PIARC 95] Permanent International Assoc. of Road Congresses, “Flexible Roads: Report of the Committee”, Proc., XX’th World Road Congress, Montreal, Sept., 1995.

[RTAC 77] Roads and Transportation Association of Canada, “Pavement Management Guide”, Ottawa, 1977.

[RTAC 86] Roads and Transportation Association of Canada, “Vehicle Weights and Dimensions Study”, Technical Committee Steering Report, Ottawa, Canada, Dec., 1986.

[Scherocman 92] Scherocman, J.A., “The Design, Construction and Performance of Stone Mastic Asphalt Pavement Layers”, Proc., Can. Tech. Asphalt Assoc., 1992.

[Shell 78] Shell International Ltd., “Shell Pavement Design Manual: Asphalt Pavements and Overlays for Road Traffic”, Shell Int. Ltd., London, 1978.

[Shell 92] Valkering, C.P. and F.D.R. Stapel, “The Shell Pavement Design Method on a Personal Computer”, Proc., Vol. 1, 7’th Int. Conf. on Asphalt Pavements, Nottingham, 1992.

[SHRP 94] Strategic Highway Research Program, “The Superpave Mix Design Manual for New Construction and Overlays”, SHRP-A-407, 1994.

[TAI 81] The Asphalt Institute Manual Series No. 1 (MS-1), “Thickness Design – Asphalt Pavements for Highways and Streets”, College Park, Maryland, Sept. 1981.

[TAI 83] The Asphalt Institute Manual Series No. 17 (MS-17), “Asphalt Overlays for Highways and Street Rehabilitation”, College Park, Maryland, June, 1983 edition.

[TAI 86] The Asphalt Institute Manual Series No. 10 (MS-10), “Soils Manual”, Lexington, Kentucky, 1986.

[TAI 91a] The Asphalt Institute Manual Series No. 1 (MS-1), “Thickness Design – Asphalt Pavements for Highways and Streets”, Lexington, Ky., Feb., 1991.

[TAC 91] Transportation Association of Canada, “Highways in Canada 1991”, Report published by TAC, Ottawa, 1991.

[TAC 94] Transportation Association of Canada, “Impacts of Canada’s Heavy Vehicle Weights and Dimensions Research and Interprovincial Agreement”, Oct., 1994.

[TAC 13] Transportation Association of Canada, “Pavement Design and Management Guide”, published by TAC, 2013.

[U.S. Army 53] Corps. of Engineers, U.S. Army, “The Unified Soil Classification System”, Waterways Experiment Station, Technical Memorandum No. 3-357, Vicksburg, Mississippi, 1953.