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In

Infrastructure Systems

PAVEMENT AND ALIGNMENT DESIGN OF A NEW RURAL ROAD IN THE PROVINCE OF BOLOGNA

Candidate:
MAHER AYAT

Supervisor:
Prof. Eng. Andrea Simone

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This thesis is dedicated to all **my Family**, to **my Father** and especially to **my Mother** who, always, encouraged me. I hope I made her proud, seeing her dream comes true.

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ACKNOWLEDGEMENTS:

I would like to express my gratitude to my supervisor Prof. Eng. Andrea Simone for providing great input in this work. He gave me the greatest opportunity by letting me doing research on such interesting topic.

A special thanks to my parents, to my two sisters and to my two brothers who were always there for me. I COULD HAVE NEVER DONE IT WITHOUT YOU.

Maher Ayat

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ABSTRACT:

This thesis aims to give a general view of pavement types all over the world, by showing the different characteristics of each one and its different life steps starting from construction, passing by maintenance and arriving until recycling phase. The flexible pavement took the main part of this work because it has been used in the last part of this thesis to design a project of a rural road. This project is located in the province of Bologna-Italy ('Comune di Argelato', 26 km in the north of Bologna), and has 5677, 81 m of length. A pavement design was made using the program BISAR 3.0 and a fatigue life study was made, also, in order to estimate the number of loads (in terms of heavy vehicles axle) to cause road's failure .

An alignment design was made for this project and a safety study was established in order to check if the available sight distance at curves respects the safety norms or not, by comparing it to the stopping sight distance.

Different technical sheets are demonstrated and several cases are discussed in order to clarify the main design principles and underline the main hazardous cases to be avoided especially at intersection. This latter, its type's choice depends on several factors in order to make the suitable design according to the environmental data. At this part of the road, the safety is a primordial point due to the high accident rate in this zone. For this reason, different safety aspects are discussed especially at roundabouts, signalized intersections, and also some other common intersection types.

The design and the safety norms are taken with reference to AASHTO (American Association of State Highway and Transportation Officials), ACT (Transportation Association of Canada), and also according to Italian norms (Decreto Ministeriale delle Starde).

Keywords: Flexible pavement, Rigid pavement, Vertical alignment, Horizontal alignment, Sight Distance (SD), Stopping Sight Distance (SSD).

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1. Introduction

1.1 Preamble:

Transportation is necessary for a nation's growth and development. In fact, it has consumed a considerable portion of human race's time and resources for as long as it has existed. Several factors should be taken into account in a pavement design, for example the traffic flow, the asphalt mixtures materials and also the environmental factors… which will define, all of them, the pavement performance.

Pavement performance is defined as the ability of a pavement to satisfactorily serve traffic over time (AASHTO, 2003). The serviceability is defined as the ability of a pavement to serve the traffic for which it was designed. Integrating both definitions will yield a new understanding of the performance which can be interpreted as the integration of the serviceability over time (Yoder and Witczack, 1975).

Usually it is required a traffic evaluation for both design and rehabilitation. Since the pavement of the new road or that under rehabilitation is usually designed for periods ranging from 10 to 20 years or more, it is to estimate or predict the design loads for this period of time accurately.

A satisfactory pavement has to respect some conditions regarding the asphalt surface that has to exhibit sufficient strength and stiffness, also, adequate sub-base layer strength to provide sufficient bearing capacity to the pavement. Moreover, a stable subgrade and adequate drainage system should be installed to eliminate moisture and avoid base layer instability. Finally a regular maintenance plan should be fixed in order to avoid the pavement deterioration.

1.2Problem Statement:

There are many pavement types all over the world. Each type has its characteristics and performances. The two most popular pavement types are the flexible pavement and the rigid one. All other types derive essentially from these two pavements. However, the pavement field has been innovative. In fact, some pavement types that are produced (Long Life Pavement LLP) and some others which still in test, were made to keep this important domain progressing to answer to new challenges.
Furthermore, a pavement life scale includes all the steps from the construction to the maintenance and recycling part. This latter became a very important field and many researches were made in order to improve pavement's quality and to save materials.

While designing a road, engineers have to pay attention not only to quotidian users but also to road special users (heavy vehicles and pedestrians). A good design is a design which assures road users comfort and safety. This latter is, obviously, the main goal of any project and still a key point in any research field.

Nowadays, many programs are used to design pavements. **BISAR 3.0** is one of the best programs used worldwide. With some required input like the number of layers, the Young's modulus of the layer and the Poison's ratio it gives the components of **stress**, **strain** and **displacement** vector.

While checking the road's safety, engineers have to check if the stopping sight distance (**SSD**) is less than the sight distance (**SD**) of the same curve or not. If yes, so the curve is safe and vehicles can pass safely. But if the stopping sight distance is greater than the sight distance, either the design speed or the curve radius has to be changed, depending on the case. Some exceptions are permitted when the curve is so short and there are no obstacles in the right hand curves.

1.3 Objectives of the thesis:

This thesis aims to:

1- Study the different types of pavements.
2- Introduce the pavement maintenance and its different categories.
3- Illustrate the pavement recycling and its different methods.
4- Make the alignment design (Horizontal alignment and vertical alignment) of a new rural road in the province of Bologna.
5- Design a pavement of a new rural road in the province of Bologna using the program BISAR 3.0.

1.4 Methodology:

The informations in this thesis were gathered from different sources: in order to study the different types of pavements and the characteristics of each one, several publications and articles were used. Other articles and books helped to understand the pavement maintenance procedure and also the pavement recycling. The AASHTO, TAC and the PIARC books were used to study the technical sheets, and other official reports helped to get informations about the project site and its characteristics. All the statistics and data used in this thesis were gathered from the official web pages.

1.5 Thesis Layout:

This thesis contains nine chapters. **Chapter 1** is an introductory chapter outlining the problem statement and the objectives of the thesis.

Chapter 2 provides a literature review. It presents a simple description of the rigid and the flexible pavement, which are the most important types of pavement from where derive all types of pavements.

Chapter 3 describes in detail the different types of pavements and the characteristics of each one. In particular it describes the flexible and the rigid pavement as all the other types of pavements derive from these two types.

Chapter 4 presents a detailed description of the pavement maintenance. The first part describes the preventive maintenance with its different treatments, and the second part explains the structural maintenance procedure and gives some solutions in order to mitigate the pavement deterioration.

Chapter 5 illustrates the different pavement recycling procedures. It takes the hot/cold in place recycling, the full depth reclamation and the hot mix recycling and describes the materials used in each procedure. Moreover, it shows the main advantages of these four procedures considering the pavement recycling as a part of the pavement's life.

Chapter 6 describes the alignment design concept and takes the main technical sheets used in the design part. In fact, it presents the horizontal alignment which comprises straight lines, circular curves and spiral curves which together can make various sequences. Moreover, it illustrates the vertical alignment which consists of straight segments connected by sag or crest vertical curves. Furthermore, it explains the sight distance concept and the precautions to be taken for a safe passing of a vehicle without hitting any object on his path. In the last part, it describes the intersections which represent an essential part of a road network, besides it gives the main factors, which depends on, the choice of the intersection type. Finally, it discusses the safety at intersections and gives some examples.

Chapter 7 describes the alignment design of a new rural road in the province of Bologna. Indeed, it presents the results of the vertical and horizontal alignment of the road. Also, it illustrates the main methods and formulas used in the design previously presented.

Chapter 8 describes the pavement design of a new rural road in the province of Bologna using the BISAR 3.0 program, which calculates the stresses, strains and displacements in an elastic multi-layer system. For this, it requires some input like the number of layers and its thickness (except the semi-infinite base layer), the young's modulus of each layer, the number of loads, the co-ordinates of the position of the center of the loads and the coordinates of the position of which output is required. In the last part, it gives some results about the pavement design (stress, strain and displacement) and some interpretations. Moreover, it studies the fatigue life of the road and gives some interpretations and recommendations.

Chapter 9 summarizes the conclusions taken from this thesis and presents recommendations for future works.

2. Literature Review:

2.1 Introduction:

Road pavements are divided into two main categories: **Rigid** and **Flexible**. The wearing surface of a rigid pavement is usually constructed of Portland cement concrete such that it acts like a beam over any irregularities in the underlying supporting material. The wearing surface of flexible pavements, on the other hand, is usually constructed of bituminous materials such that they remain in contact with the underlying material even when minor irregularities occur (Traffic and highway engineering, Nicholas J. Garber and Lester A. Hoel 1999).

Generally, flexible pavements are constructed of a bituminous surface underlaid with a layer of granular material and a layer of fine materials. However, rigid pavements consist of Portland cement concrete and may or may not have a base course between the subgrade and the concrete surface.

2.2 Structural components of a flexible pavement:

Flexible pavements consist of a subgrade (prepared roadbed), the sub-base, the base and the wearing surface. This latter, when made of Hot Mix Asphalt becomes stiffer and contribute more to the pavement strength.

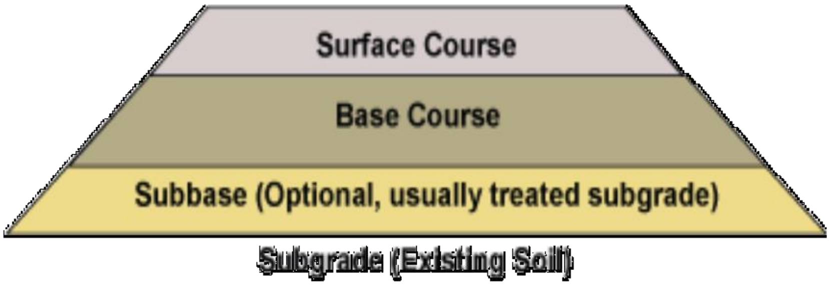


Figure 2.1: Schematic of a Flexible Pavement

→The performance of the pavement depends on the satisfactory performance of each component.

2.2.1 Subgrade:

The subgrade is the natural material located along the horizontal alignment of the pavement (It serves as the foundation of the pavement structure). Depending on the type of pavement being constructed, it is necessary to treat the subgrade material to achieve the required the strength properties.

2.2.2 Sub-base course:

Located immediately above the subgrade, the sub-base component consists of material of a superior quality to that which generally is used for subgrade construction. When the quality of the subgrade material meets the requirements of the sub-base material, the sub-base component may be omitted (Traffic and highway engineering, Nicholas J. Garber and Lester A. Hoel 1999).

→When the sub-base material does not correspond to the requirements, a process of treating soils to improve their engineering properties known as stabilization can be used. In fact, the available material should be treated with other materials to achieve the necessary properties.

2.2.3 Base course:

The base course is placed above the sub-base (above the subgrade if the sub-base course is not used). It consists of granular materials such as sand, crushed stone, crushed or uncrushed gravel and crushed or uncrushed slag.

Usually, the base course materials include stricter requirements than those for sub-base course. In some cases, to increase the stiffness characteristics of heavy-duty pavements, the base course can be treated with asphalt or Portland cement.

2.2.4 Surface course:

The surface course is the upper layer of the pavement section located immediately above the base course. The surface course in flexible pavements consists generally of a mixture of mineral aggregates and asphaltic materials. It must be able to withstand a wide variety of factors that can accelerate the deterioration process of the pavement.

2.3 Flexible Pavement Distress and Performance:

2.3.1 Flexible Pavement Distress:

The most commonly distresses on flexible pavement surfaces include cracking, rutting, pothole, and surface deterioration. In this section, various pavement distresses classes are briefly discussed according to the definitions from the US Department of Transportation distress identification manual (Federal Highway Administration, 2003) and the LTTP Distress Identification Manual.

2.3.1.1 Fatigue Cracking:

Fatigue (also called alligator) cracking, which is caused by fatigue damage, is the principal structural distress which occurs in asphalt pavements with granular and weakly stabilized bases. Alligator cracking first appears as parallel longitudinal cracks in the wheel paths, and progresses into a network of interconnecting cracks resembling chicken wire or the skin of an alligator. Alligator cracking may progress further, particularly in areas where the support is weakest, to localized failures and potholes.

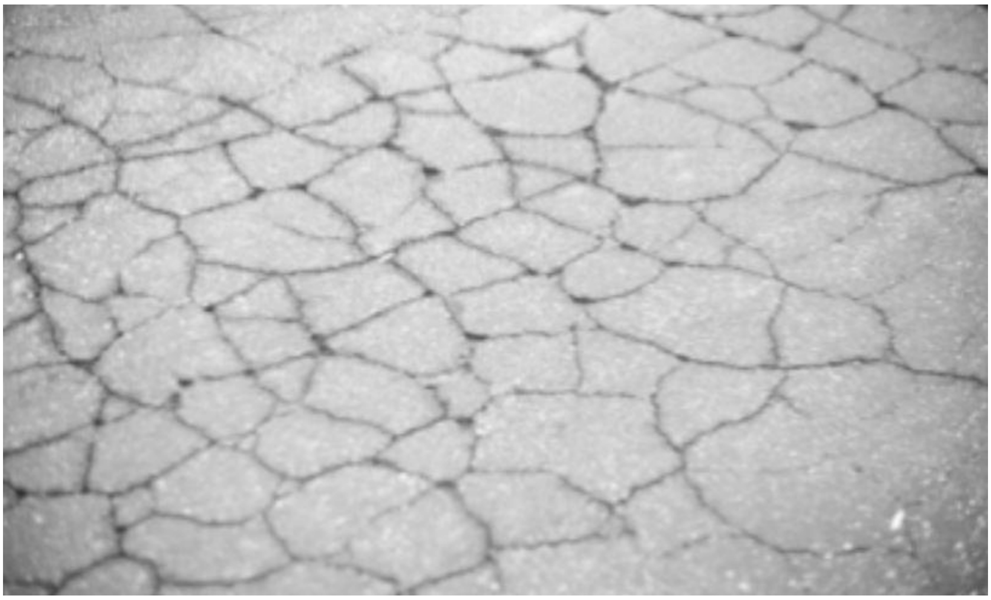


Fig 2.2: Fatigue (alligator) cracking in Flexible Pavement

Factors which influence the development of alligator cracking are:

●The number and magnitude of applied loads;
●The structural design of the pavement (layer materials and thicknesses);
●The quality and uniformity of foundation support;
●The asphalt content.

2.3.1.2 Block Cracking and Transverse (Thermal):

Block cracking is the cracking of an asphalt pavement into rectangular pieces ranging from approximately 30 cm to 300 cm on a side. Block cracking occurs over large paved areas such as parking lots, as well as roadways, primarily in areas not subjected to traffic loads, but sometimes also in loaded areas. Thermal cracks typically develop transversely across the traffic lanes of a roadway.

Figure 2.3: Longitudinal Cracking (Medium Severity)

Block cracking and thermal cracking are both related to the use of asphalt which is or has become too stiff for the climate. Both types of cracking are caused by shrinkage of the asphalt in response to low temperatures, and progress from the surface of the pavement downward. The key to minimizing block and thermal cracking is using an asphalt of sufficiently low stiffness (high penetration).

2.3.1.3 Potholes:

A pothole is a bowl-shaped hole through one or more layers of the asphalt pavement structure, between about 15 and 90 centimeters in diameter. Potholes begin to form when fragments of asphalt are displaced by traffic wheels, e.g., in alligator-cracked areas. Potholes grow in size and depth as water accumulates in the hole and penetrates into the base and subgrade, weakening support in the vicinity of the pothole.

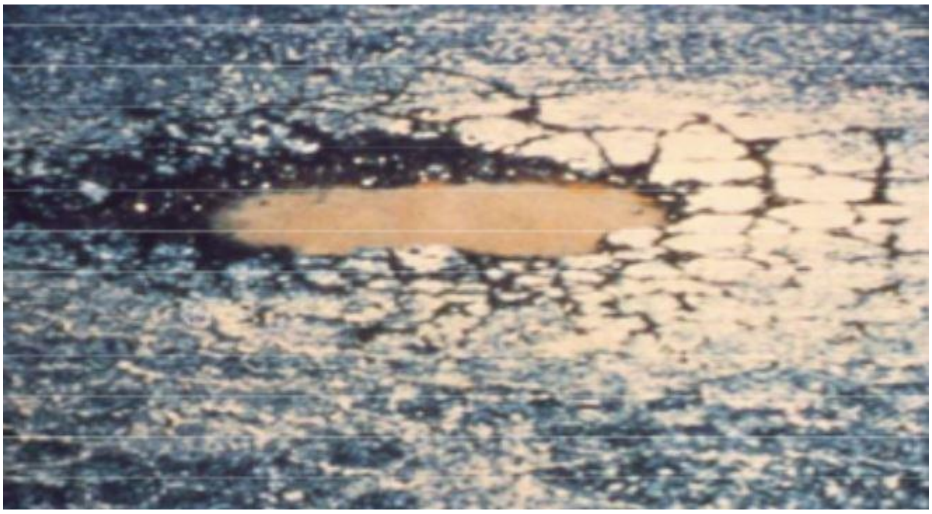


Figure 2.4: High Severity Pothole

2.3.1.4 Rutting:

Rutting is the formation of longitudinal depression of the wheel paths, most often due to consolidation or movement of material in either the base or subgrade or in the asphalt layer. Another, unrelated, cause of rutting is abrasion due to studded tires and tire chains. Deformation which occurs in the base and underlying layers is related to the thickness of the asphalt surface, the thickness and stability of the base and sub-base layers, and the quality and uniformity of subgrade support, as well as the number and magnitude of applied loads.

Figure 2.5: Rutting

2.3.1.5 Longitudinal Cracking:

Non-wheel path longitudinal cracking in an asphalt pavement may reflect up from the edges of an underlying old pavement or from edges and cracks in a stabilized base, or may be due to poor compaction at the edges of longitudinal paving lanes. Longitudinal cracking may also be produced in the wheel paths by the application of heavy loads or high tire pressures.

Figure 2.6: Longitudinal Cracking (Medium Severity)

2.3.2 Stress Distribution:

Stress distribution in a road structure is studied in order to know how phenomena develop in the road structure and in particular to determine the behavior of the asphalt layers. Figure 2.7 shows a wheel load applying a downward pressure on a road surface. The load is spread out and reduced in intensity by the various pavement layers. The pressure, P1, on the subgrade is much less than the tire pressure, P0, on the pavement surface. Consequently, higher quality – and generally more expensive – materials are used in the more highly stressed upper layers of all pavement systems, and lower quality and less expensive materials are used for the deeper layers of the pavement. However, if this condition is not met or if the deterioration of these layers occurs, the durability of the pavement is significantly reduced. This may result in the onset of premature pavement distress, in the form of fissures, longitudinal cracks, alligator cracks, surface depreciations or potholes.

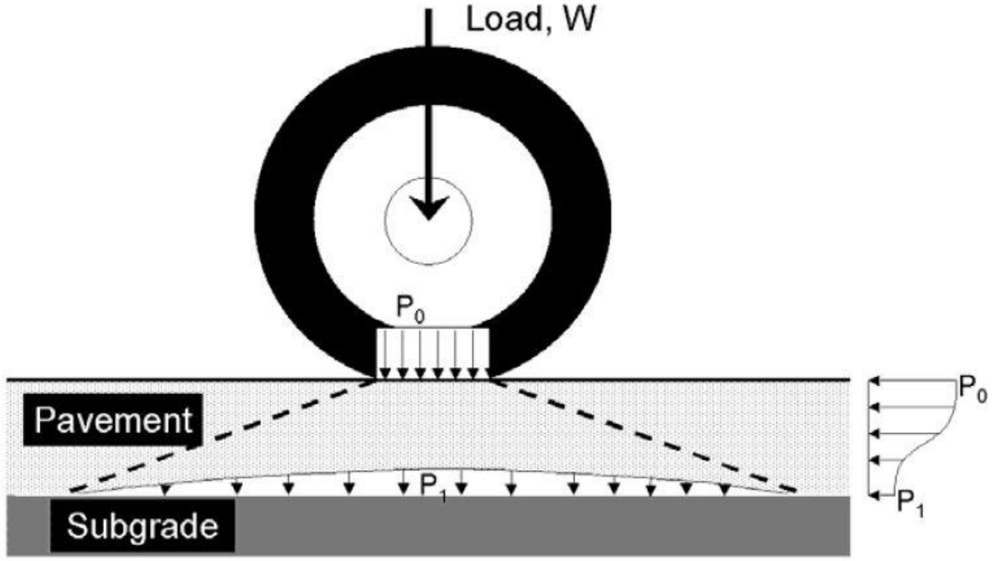


Figure 2.7: Spread of wheel load pressure through the pavement structure

The vertical forces are due to the weight applied on the wheel, the horizontal shear stresses, occurring on the pavement surface during accelerating and breaking phases are due to the adherence between tires and wear course during rolling. The upper layers are subjected to bending stresses while the lower layers are subjected to vertical forces, mainly compression. Numerous studies undertaken in recent years have revealed that, indeed, stress distribution is very complex because the tensile progress does not only affect surfaces layers, but also the layers immediately below (Muraya, 2007).

It is possible to represent the state of compression and tension (Figure 2.8 below): The load applied by a tire can be illustrated with a concentrated force that determines a compression zone, immediately below, and two traction areas in the adjacent sides.

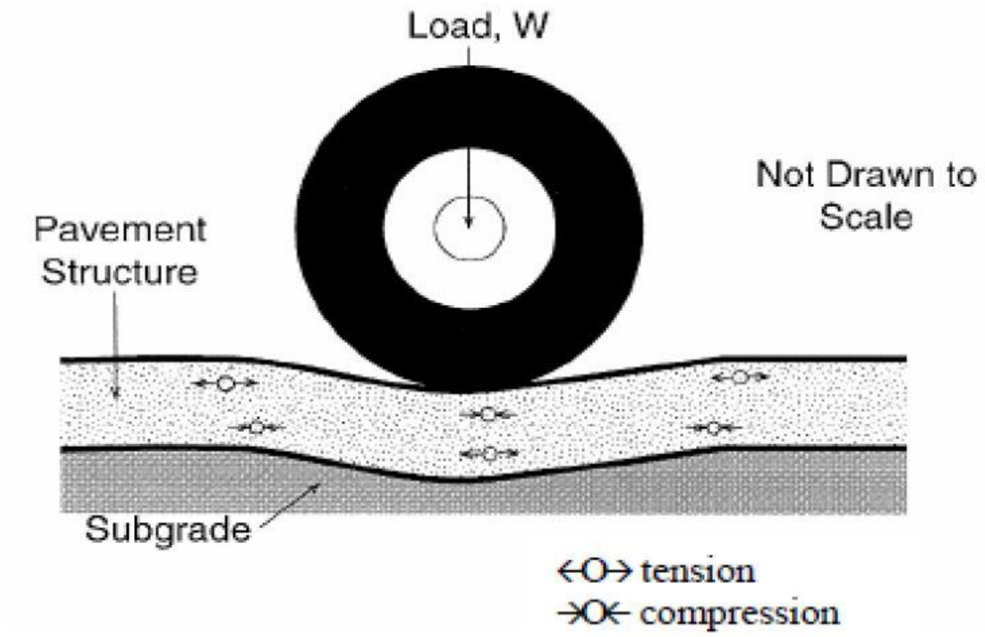


Figure 2.8: Pavement Deflection under Load

If the deflection is large enough and occurs enough times, the tension stress can cause a fatigue crack at the bottom of the layer. Additional loads cause this crack to migrate upward until it reaches the surface. Surface water can then penetrate through the crack into the base and weaken it. This causes larger deflections in the adjacent pavement and more cracks until pavement failure (alligator cracking) occurs.

3. Pavement Analysis:

3.1 Introduction:

Generally, Pavements are divided into two main categories: **Rigid** and **Flexible**. The wearing surface of rigid pavements is usually constructed of Portland cement concrete such that it acts like a beam over any irregularities in the underlying supporting material. The wearing surface of flexible pavements, on the other hand, is usually constructed of bituminous materials such that they remain in contact with the underlying material even when minor irregularities occur. Flexible pavements usually consist of a bituminous surface underlaid with a layer of granular material and a layer of a suitable mixture of coarse and fine materials. Traffic loads are transferred by the wearing surface to the underlying supporting materials through the interlocking of aggregates, the frictional effect of the granular materials and the cohesion of the fine materials.

Flexible pavements are further divided into three subgroups: High type, intermediate type and low type. High type pavements have wearing surfaces that adequately support the expected traffic load without visible distress due to fatigue and are not susceptible to weather conditions. Intermediate type pavements have wearing surface that range from surface treated to those with qualities just below that of high type pavements. Low type pavements are used mainly for low cost roads and have wearing surfaces that range from untreated to loose natural materials to surface-treated earth. (Traffic and highway engineering, 1999).

3.2 Types of Pavements:

There are several kinds of pavement there can be used for multiple purposes. It often happens that some pavements can be used for more than one type of vehicle/transportation or load, but often only few are suitable for the purpose they are designed for.

***3.2.1 Flexible Pavement*:**

A flexible pavement structure is typically composed of several layers of material with better quality materials on top where the intensity of stress from traffic loads is high and lower quality materials at the bottom where the stress intensity is low. Flexible pavements can be analyzed as a multi-layer system under loading. A typical flexible pavement structure consists of the surface course and underlying base and sub base courses. Each of these layers contributes to structural support and drainage. When hot mix asphalt (HMA) is used as the surface course, it is the stiffest and may contribute the most to pavement strength, which is depending on the thickness. The underlying layers are less stiff, but are still important to pavement strength as well as drainage and frost protection.

When a seal coat is used as the surface course, the base generally is the layer that contributes most to the structural stiffness. A typical structural design results in a series of layers that gradually decrease in material quality with depth.

Figure 3.1 shows a typical section for a flexible pavement.

Figure 3.1: Typical section for a flexible pavement

<u>3.2.2 Rigid Pavement</u>:

A rigid pavement structure is composed of a hydraulic cement concrete surface course and underlying base and sub base courses (if used). Another term commonly used is Portland cement concrete (PCC) pavement, although with today's pozzolanic additives, cements may no longer be technically classified as "Portland". The surface course is the stiffest layer and provides the majority of strength. The base or sub base layers are orders of magnitude less stiff than the PCC surface but still make important contributions to pavement drainage and frost protection and provide a working platform for construction equipment.

Rigid pavements are substantially 'stiffer' than flexible pavements due to the high modulus of elasticity of the PCC material, resulting in very low deflections under loading. The rigid pavements can be analyzed by the plate theory. Rigid pavements can have reinforcing steel, which is generally used to handle thermal stresses to reduce or eliminate joints and maintain tight crack widths.

Figure 3.2 shows a typical section for a rigid pavement.

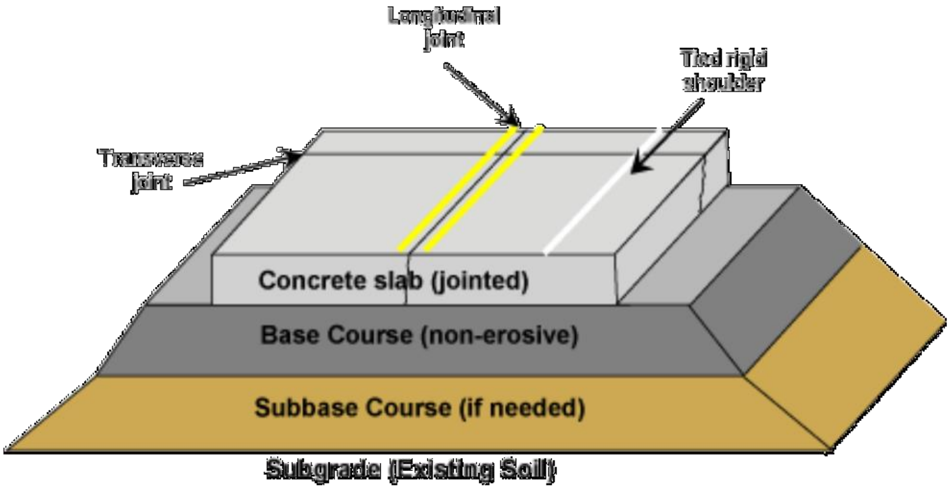


Figure 3.2: Typical section for a rigid pavement

3.2.3 Composite Pavement:

A composite pavement is composed of both hot mix asphalt (HMA) and hydraulic cement concrete. Typically, composite pavements are asphalt overlays on top of concrete pavements. The HMA overlay may have been placed as the final stage of initial construction, or as part of a rehabilitation or safety treatment. Composite pavement behavior under traffic loading is essentially the same as rigid pavement.

3.2.4 Perpetual Pavement:

Perpetual pavement is a term used to describe a long-life structural design. It uses premium HMA mixtures, appropriate construction techniques and occasional maintenance to renew the surface. Proper construction techniques need to be kept in mind to avoid problems with permeability, trapping moisture, segregation with depth, and variability of density with depth. A perpetual pavement can last 30 yr. or more if properly maintained.

Structural deterioration typically occurs due to either classical bottom-up fatigue cracking, rutting of the HMA layers, or rutting of the subgrade. Perpetual pavement is designed to withstand almost infinite number of axle loads without structural deterioration by limiting the level of load-induced strain at the bottom of the HMA layers and top of the subgrade and using deformation resistant HMA mixtures.

Figure 3.3 shows a generalized perpetual pavement design.

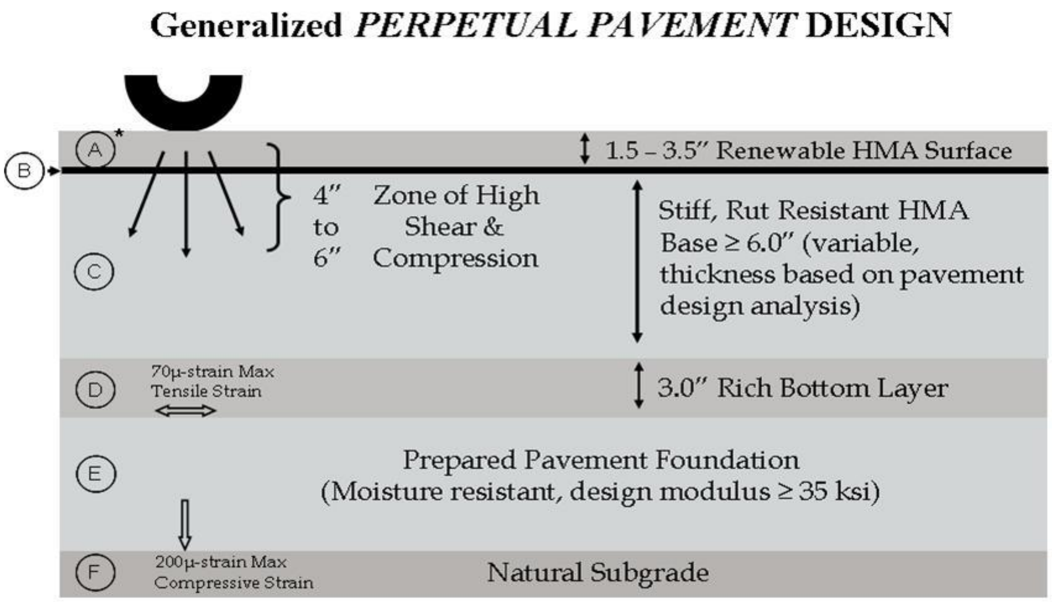


Figure 3.3: Generalized perpetual pavement design

***<u>3.2.5 Continuously Reinforced Concrete Pavement</u>*:**

CRCP provides joint-free design. The formation of transverse cracks at relatively close intervals is a distinctive characteristic of CRCP. These cracks are held tightly by the reinforcement and should be of no concern as long as the cracks are uniformly spaced, do not spall excessively, and a uniform non-erosive base is provided.

Figure 3.4 shows a typical section of CRCP.

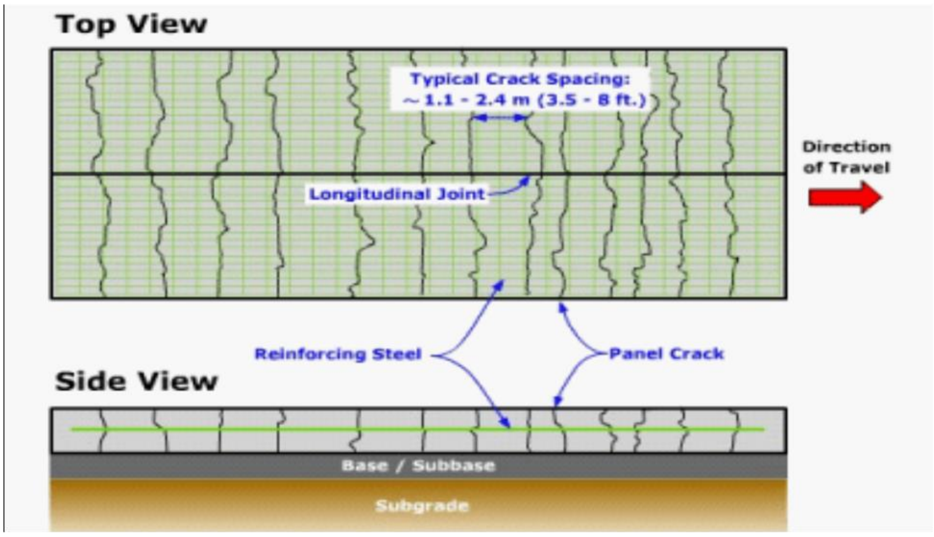


Figure 3.4: Continuously Reinforced Concrete Pavement.

***<u>3.2.6 Concrete Pavement Contraction Design (CPCD)</u>*:**

CPCD uses contraction joints to control cracking and does not use any reinforcing steel. An alternative designation used by the industry is jointed concrete pavement (JCP). Transverse joint spacing is selected such that temperature and moisture stresses do not produce intermediate cracking between joints. Dowel bars are typically used at transverse joints to assist in load transfer. Tie bars are typically used at longitudinal joints.

Figure 3.5 shows a typical section of CPCD.

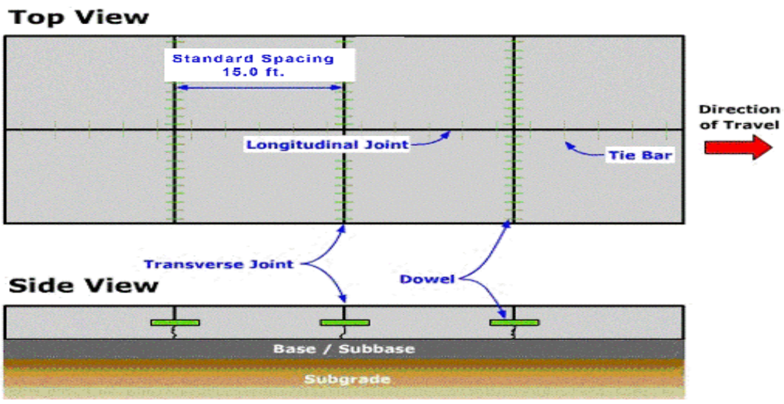


Figure 3.5: Concrete Pavement Contraction Design (CPCD)

3.2.7 Jointed Reinforced Concrete Pavement (JRCP):

JRCP uses contraction joints and reinforcing steel to control cracking. Transverse joint spacing is longer than that for concrete pavement contraction design. (CPCD) This rigid pavement design option is no longer endorsed by the department because of past difficulties in selecting effective rehabilitation strategies. However, there are several remaining sections in service.

Figure 3.6 shows a typical section of jointed reinforced concrete pavement.

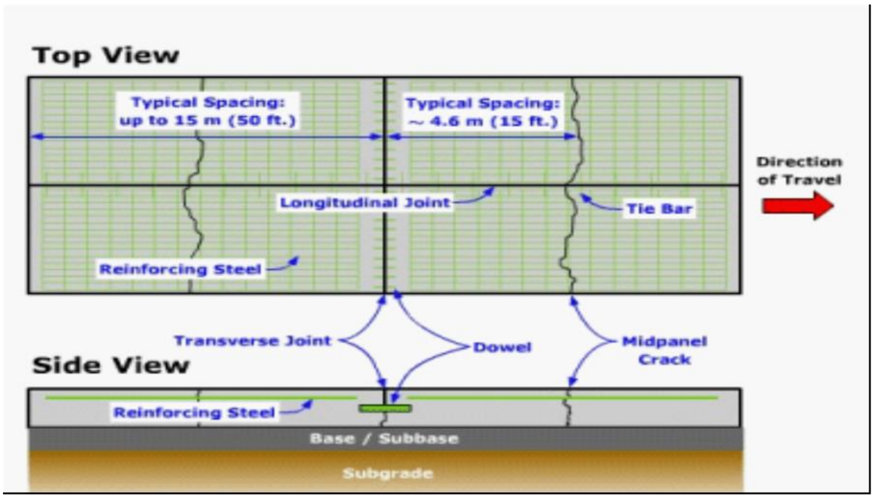


Figure 3.6: Jointed Reinforced Concrete Pavement (JRCP)

3.2.8 Post-tensioned Concrete Pavement:

Post-tensioned concrete pavements remain in the experimental stage and their design is primarily based on experience and engineering judgment. Post-tensioned concrete has been used more frequently for airport pavements than for highway pavements because the difference in thickness results in greater savings for airport pavements than for highway pavements.

3.3 Rigid and flexible pavement characteristics:

The primary structural difference between a rigid and flexible pavement is the manner in which each type of pavement distributes traffic loads over the subgrade. A rigid pavement has a very high stiffness and distributes loads over a relatively wide area of subgrade – a major portion of the structural capacity is contributed by the slab itself.

The load carrying capacity of a true flexible pavement is derived from the load-distributing characteristics of a layered system.

Figure 3.7 shows load distribution for a typical flexible pavement and a typical rigid pavement.

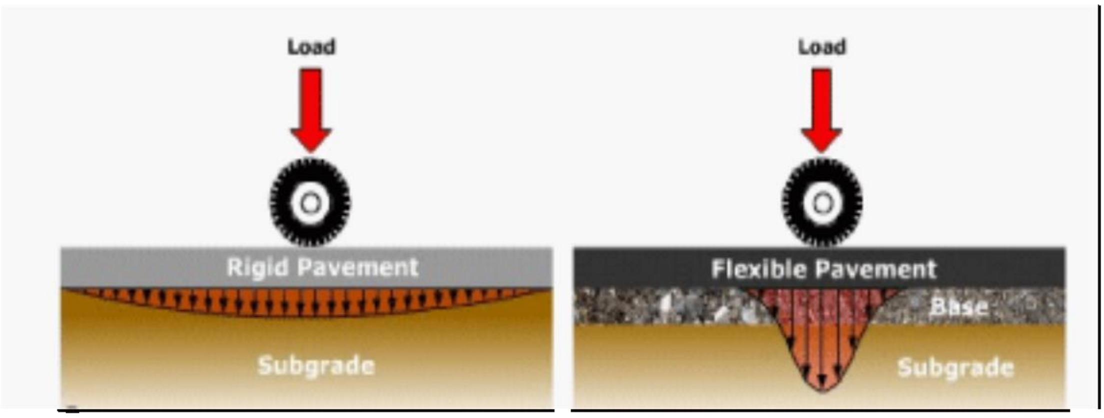


Figure 3.7: Typical stress distribution under a rigid and a flexible pavement.

4. Pavement Maintenance:

4.1 Introduction:

An asphalt pavement, when designed and constructed properly, will provide years of service. All pavements will eventually require some type of maintenance. Thus, maintenance is an essential practice in providing for the long-term performance and the esthetic appearance of an asphalt pavement.

The purpose of pavement maintenance is to correct deficiencies caused by distresses and to protect the pavement from further damage. Moreover, various degrees or levels of maintenance can be applied to all pavements, regardless of the end use. Pavement maintenance is either preventive or corrective. Preventive maintenance is the procedure performed to protect the pavement and decrease the rate of deterioration of the pavement quality. Corrective maintenance is the procedure performed to correct a specific pavement failure or area of distresses. Some procedures will address both functions (Roberts et al. 1991). The sealing of cracks for the most part is considered a preventive maintenance measure. Patching is considered a corrective maintenance measure.

Pavement maintenance can be described by two different categories:

●*Preventive maintenance*: Activities that prevent or reduce further damage to the pavement

●*Structural maintenance:* Activities that repair or improve the structural integrity of the pavement (Very expensive).

4.2 Preventive Maintenance:

Preventive maintenance activities include:

● Pavement sealers or sealcoats

● Crack filling and sealing

● Surface Treatments

4.2.1 Pavement sealers or sealcoats:

Pavement sealers are used to restore or rejuvenate an oxidized asphalt pavement surface. They are also used to fill hairline cracks that are less than 3 mm wide.

Some sealers provide an improved or 'new' appearance to an aged asphalt pavement and can protect the asphalt pavement from fuel or oil damage.

The most common pavement sealers are:

- Fog seals
- Asphalt emulsion seal coat
- Coal tar seal coat

4.2.2 Crack filling and sealing:

The most common and widely used maintenance activity for pavements, regardless of use, is crack sealing or filling. Crack sealing and filling is an inexpensive maintenance procedure that will significantly delay further deterioration of the pavement.

Cracks less than 3 mm wide are too narrow to be sealed or filled. A pavement sealer or surface treatment is adequate to treat these narrow cracks. Cracks that are 3-25 mm can be sealed or filled with an application of a crack sealant or filling material. However, cracks that are greater than 25 mm wide are generally too wide to be sealed or filled and should be repaired through the use of a patching mixture or they should be cut out and replaced with a full depth patch.

Crack characteristics	*Crack repair procedure*	
	Sealing	*Filling*
Crack width, mm	4–19	4–25
Annual horizontal movement (mm)	≥2.5	≤2.5
Crack type	Transverse Reflective Longitudinal	Longitudinal Widely spaced block

Table 4.1: Sealing or Filling Cracks

Crack sealing and crack filling are actually two separate procedures:

- "Crack sealing" is the installation of a specially formulated crack sealing material either above or into **working cracks** using unique configurations to prevent the intrusion of water into the crack.
- "Crack filling" is the placement of crack filling material into **non-working** cracks to substantially reduce the intrusion of water into the wrack.

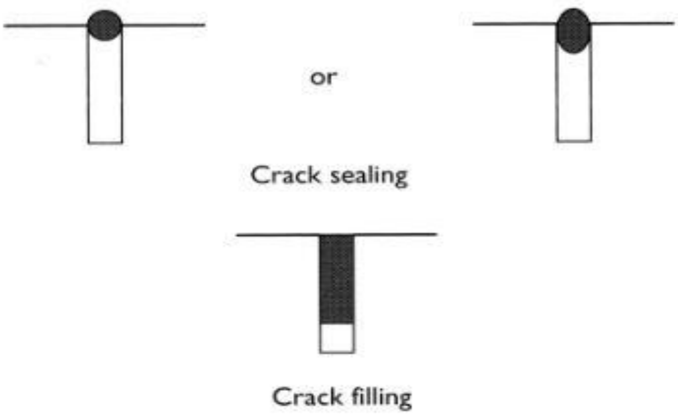


Figure 4.1: Crack Sealing and Filling differences

→The significant differences are that crack sealing is applied to working cracks and crack filling is applied to non-working cracks. Moreover, crack sealing involves placing sealing material in or on top of the crack, but crack filling involves placing filling material in the crack.

4.2.3 Surface treatments:

Surface treatment is a broad category, encompassing several types of asphalt and coal tar sealers or asphalts aggregate combinations. An asphalt surface treatment consists of a thin layer of asphalt concrete formed by the application of an asphalt emulsion, cutback or asphalt binder plus aggregate to protect or restore an existing pavement surface.

The surface treatment will perform one or more of the following functions:

- Provide a weather resistant surface.
- Provide a fuel or oil resistant surface.
- Provide an esthetically pleasing coating to the pavement surface.
- Fill or seal hairline or cracks under 3 mm width.
- Fill distortions or rutting.
- Provide a skid resistant surface.

→ One function that a surface treatment will not provide is structural strength. The luck of any significant aggregate interlock and thickness in a surface treatment results in no structural strength and is not considered when determining the overall required thickness for an asphalt pavement.

4.3 Structural Maintenance:

Structural maintenance activities include:

●Patching

●Pothole filling

●Overlays (Resurfacing)

●Reconstruction

<u>4.3.1 Patching:</u>

The patching of a pavement is a permanent solution to pavement distresses, usually high severity distresses. The purpose of patching is essentially, permanently repairing the portion of the pavement that is defective due to:

- Pavement distress, such an alligator cracking, severe transverse cracking, severe block cracking, etc.
- Repair a cut in the pavement due to a utility cut or repair.

Figure 4.2: Pavement Distress Patch

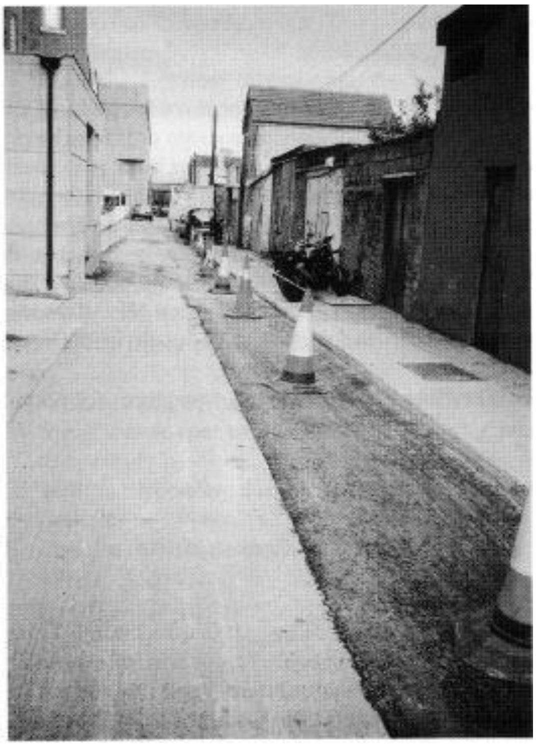


Figure 4.3: Utility Cut Patch

4.3.2 Pothole filling:

Pothole filling is a somewhat temporary measure, which can also be considered patching. The purpose of this method is to temporarily eliminate the pothole as a road hazard and nuisance. Potholes are the result of a rapid disintegration portion of the pavement that was not repaired in time.

Potholes can occur on any type of asphalt pavement, including local roads, parking lots, and freeways. Pothole filling can be part of a routine maintenance program or it may be performed as an emergency repair. Potholes can be a hazard to vehicles and pedestrians.

Figure 4.4: Pothole Hazard

Potholes generally become more apparent under harsh weather conditions, such as freezing or very wet weather. Thus, emergency pothole filling usually occurs during inclement weather.

Pothole filling is a very simple process. There are four recognized procedures for pothole filling:

- Throw and go

- Throw and roll

- Semi permanent

- Injection

4.3.3 Overlays (Resurfacing):

In more severe cases of asphalt failure, a long-term and cost-effective solution is to resurface the asphalt pavement. If there is a grade depression (standing water on the pavement) and/or large sections of alligatored areas (interconnecting cracks forming a series of blocks resembling an alligator's skin), it is a good idea to have the pavement resurfaced. This process consists of several steps; In fact, after preparation and cleaning of the area, tack coat will be applied. Hot asphalt will then be installed to the approximate specified depth and compacted with a multi-ton vibratory roller to guarantee proper compaction.

***Resurfacing options:**

a) *Geotextile Reinforced Resurfacing:* An option that may be included with asphalt resurfacing is Petromat. Petromat is a non-woven, petroleum-based geotextile fabric used to retard reflective cracking between the existing pavement and the newly installed asphalt surface. This fabric acts as a waterproofing membrane, while also adding structural support and strength.

b) *Leveling Binder:* In low areas, hot asphalt is installed at various depths to adjust pitch to proper grades.

c) *Butt/Joint grinding:* In areas requiring the resurface to tie into other existing surfaces (i.e., concrete, etc.) asphalt is removed along the perimeter to allow proper depth of asphalt on the edge.

d) *Transitional milling:* In areas requiring the resurface to tie into other existing surfaces (i.e., concrete, etc.), asphalt will be milled and replaced to allow proper depth and transitions. An asphalt milling machine is used to remove an appropriate depth of pavement in a grinding process. The spoils can then be hauled off and recycled.

4.3.4 Reconstruction:

As asphalt pavement progresses through its performance lifecycle, its appearance diminishes over time. Fine hairline cracks spread and deepen within the asphalt. Without ongoing maintenance, water may enter through cracks and holes may form, undermining the substrate. In this case, the most effective form of repair is to remove and replace the deteriorated area.

This process consists of several important steps to ensure that the repair is performed properly: Area(s) will be milled and the existing deteriorated asphalt will be removed to the approximate specified depth. Existing stone base will be compacted and tack coat will be applied to perimeter of patches to guarantee proper bonding. Hot asphalt will be installed and compacted with a multi-ton vibratory roller and/or vibratory plate.

→ the cost for asphalt removal & replacement depends upon the geographic location, the amount of grading and substrate work required, and other site-specific factors.

5. Pavement Recycling:

5.1 Introduction:

The use of recycled materials in asphalts pavements has been occurring with varying degrees of success for the past 20 years. RAP reclaimed concrete pavement, coal fly ash, and the blast furnace slags are the most common materials that are recycled back into an asphalt pavement (United States Department of Transportation USDOT 2000).

Asphalt pavement recycling is the recycling or reusing an existing asphalt pavement into a new and structurally sound asphalt pavement. There are four common methods used in asphalt pavement recycling:

●Cold in-place recycling

●Hot in-place recycling

●Full Depth Reclamation

●Hot Mix Recycling

5.2 Cold in-place recycling:

Cold in-place recycling involves the recycling of existing asphalt pavement in situ without the use of heat. An asphalt emulsion is typically used as recycling agent. The process includes pulverizing or tilling the existing pavement, the application of the recycling agent, placement and compaction.

The use of a recycling train is often used for large or long roadway projects. The recycling train consists of pulverizing, screening, crushing and mixing units. The processed roadway is deposited in a window from the mixing machine, where it is then picked up, placed and compacted with conventional HMA paving equipment.

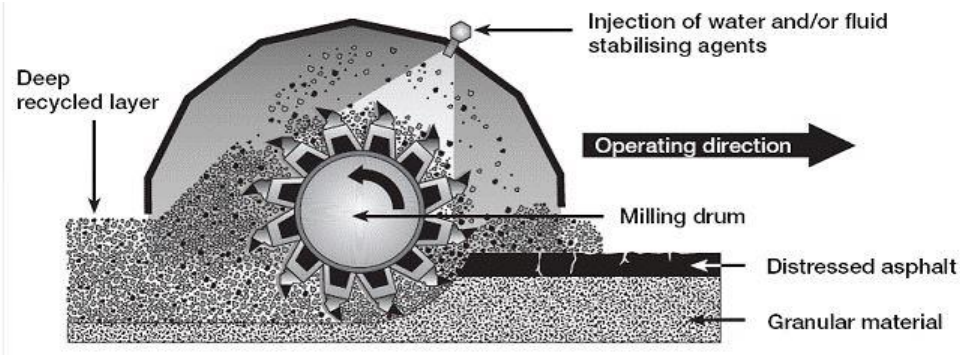


Figure 5.1: Cold in-place recycling

→Advantages of cold in-place recycling include significant remedial corrections of most pavement distresses, environmentally friendly, and the complete reuse of the existing pavement.

5.3 Hot in-place recycling:

Hot in-place recycling involves heating and softening the existing asphalt pavement and then scarifying it. A recycling agent, usually an asphalt emulsion, is added to the sacrificed RAP. Sometimes, a new asphalt mixture is also added to the RAP. The depth of recycling can vary from 20 to 50 mm.

Hot in-place recycling can be performed in either a single pass or a multiple pass operation. In a single pass operation, the sacrificed RAP is combined with a new asphalt mixture if desired, and then compacted. In a multiple pass operation, the RAP and recycling agent is compacted first and then a new wearing course is added.

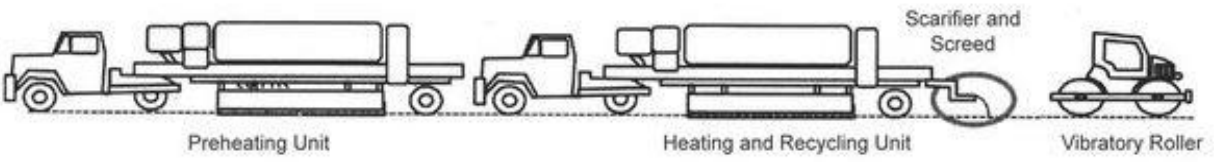


Figure 5.2: Hot in-place recycling Train

→The advantages of hot in-place recycling are that pavement distresses, including surface cracks can be corrected and the existing oxidized asphalt binder can be rejuvenated.

5.4 Full depth reclamation:

Full in depth reclamation is where the entire asphalt pavement and a predetermined amount of the underlying base course are treated to produce a stabilized base course. The existing asphalt pavement course becomes part of the new pavement's base course. Full depth reclamation is a cold mix recycling process using asphalt emulsions, calcium chloride, fly ash and possibly other additives to stabilize the base course.

The process consists of pavement pulverization, mixing with the additives, compaction, and the construction of a wearing course. If the existing pavement material is not adequate to provide the desired thickness of stabilized base, new aggregates may be added. Full depth reclamation is typically performed to depths of 100-300 mm.

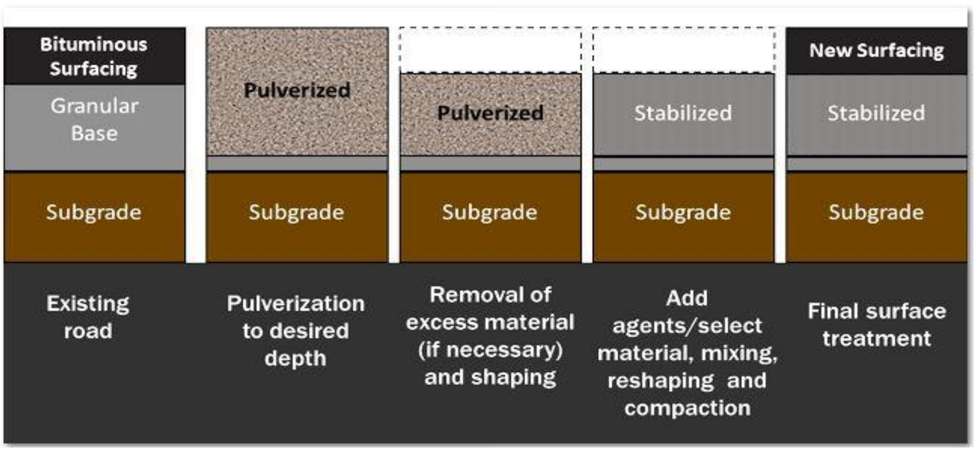


Figure 5.3: Full depth reclamation process

→This method has an advantage that it can remove most of the pavement distresses and upgrade its structural strength.

5.5 Hot mix recycling:

Hot mix recycling involves removing or milling up the existing asphalt pavement, crushing it if necessary, and adding it to HMA at mixing plant. RAP can be added at both batch mixing and drum plants. The HMA containing RAP is constructed using the same methods for conventional asphalt mixtures (United States Department of Transportation USDOT 1997).

The most common method of pavement recycling is the hot mix recycling method. The use of hot mix recycling is prevalent to geographical areas with some areas using RAP in all the HMA being produced and some areas with no RAP usage at all. The availability of RAP and landfills usually determine how many tones of HMA being produced contain RAP. Urban areas generally see more RAP usage than rural areas.

During the milling or crushing process of the existing asphalt pavement, a significant amount of fine material can be generated, which can limit the amount of RAP used in the new asphalt mixture. The heating of the RAP in the mixing plant extracts most of the asphalt binder from it and allows it to be blended throughout the new asphalt mixture. The ability of the mixing plant to transfer enough heat to dry the RAP and extract the asphalt binder from it limits the total amount of RAP that can be incorporated in the HMA.

Some modern drum mixing plants, such as double drum plants, have been designed to incorporate up to 70 percent RAP or more, but typically the maximum is 50 percent. Batch mixing plants can usually only incorporate up to 30 percent RAP in the HMA.

6. Technical Sheets: Alignment Design Concept

6.1 Horizontal Alignment:

6.1.1 General Principles:

Generally, the horizontal alignment of a road may comprise straight lines, circular curves (with a constant radius), and spiral curves, whose radius changes regularly to allow for a gradual transfer between adjacent road segments with different curve radii. Various sequences of these three components are possible.

Figure 6.1 shows three types of common sequences: Simple curve, curve with spiral(s), and curves made up of several decreasing radii.

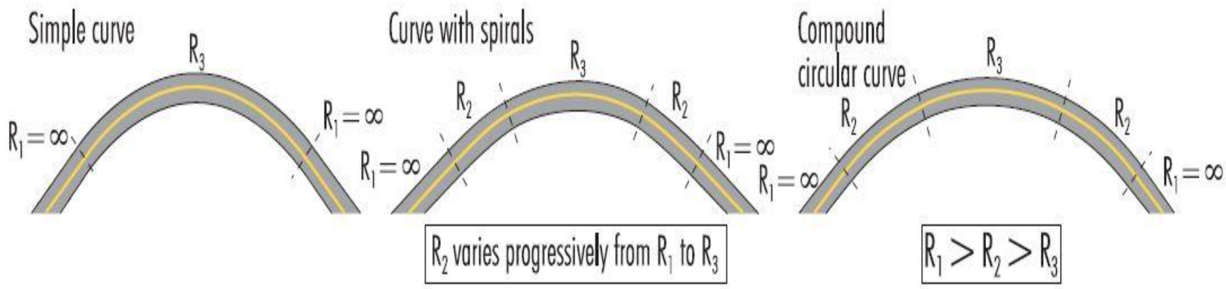


Figure 6.1: Examples of horizontal alignment components

● **Accidents:** Several studies have shown that the accident risk in curves is so much high. It can be concluded that:

-The accident rate in curves is 1.5 to 4 times higher than in tangents (Zegeer et al, 1992)

-The severity of accidents in curves is high (Glennon et al, 1986). From 25% to 30% of fatal accidents occur in curves (Lamm et al, 1999).

-Approximately 60% of all accidents to occur in horizontal curves are single-vehicle off-road accidents (Lamm et al, 1999).

-The proportion of accidents on wet surfaces is high in horizontal curves.

-Accidents occur primarily at both ends of curves. Council (1998) notes that in 62% of fatalities and 49% of other accidents occurring in curves, the first manoeuvre that led to the accident was made at the beginning or the end of the curve.

→Thus, there is a relative relationship between the speed reduction in the curve and the probability of accident: When the required speed reduction in the curve is high, the probability of error and accident becomes also so high (encroachment, skidding, run-off- the road, etc.). Moreover, the risk is even higher when the speed reduction is unexpected or unusual (isolated sharp curve).

Spacek (2000) tried to describe the behavior of drivers in curves by six track types (Figure 6.2). The correcting crack type which is due to an underestimation of the curve features locally reduces the radius followed by the car and increases the risk of accident. Improvements of sight distance, curve conspicuity and warning devices may reduce this type of problem.

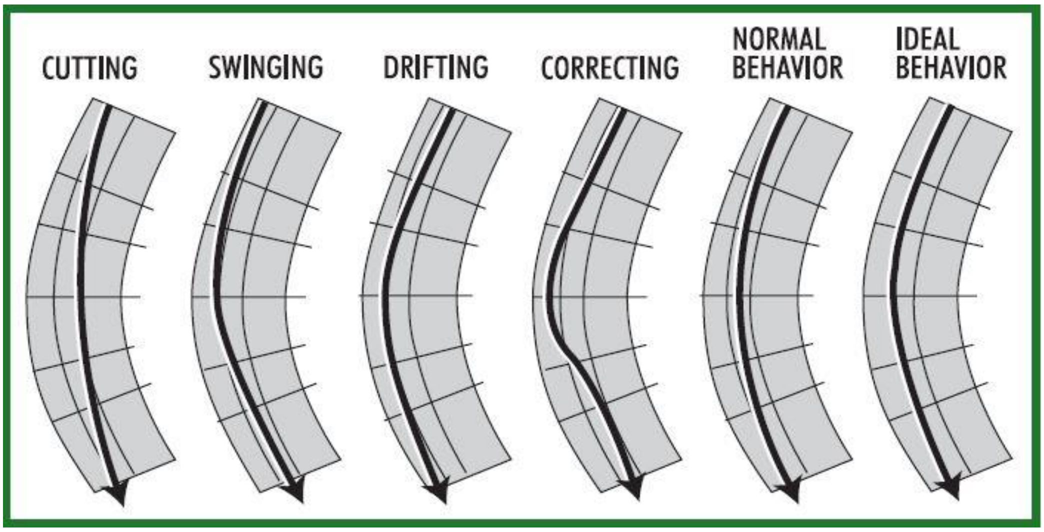


Figure 6.2: Six track types in curves

● **Potential Solutions:** Lengthening the radius of a curve is a solution that is often considered to reduce accidents in sharp horizontal curves. However, costs can be very high and the economic effectiveness of the measures needs to be properly assessed.

Other potential solutions include:

-Improving warning and guidance provided for drivers: better sight distance, curve conspicuity, signing and marking, delineation.

-Minor geometric improvements, including modifications to the shoulder and roadside conditions.

6.1.2 Curve Radius (or Degree of Curve):

From a stability point of vue, vehicle travelling in a curve is always pushed toward the exterior side of the road by the centrifugal force. This latter is counteracted by transverse friction (between the tires and the road) and the superelevation force (Figure 6.3).

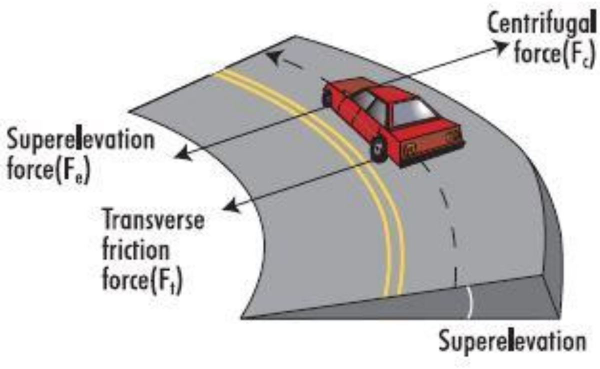


Figure 6.3: Curve-System of Forces

The higher is the vehicle speed, the bigger is the magnitude of the centrifugal force until reaching a point where this force is equal to the sum of these counteracting forces and skidding occurs:

Fc = Fe + Ft (Equation 1)

Where: Fc = Centrifugal Force

Fe = Superelevation Force

Ft = Transverse Elevation Force

→However, some vehicles having a high center of gravity may overturn before skidding.

By Transforming equation 1, one can derive the basic equation that is used to calculate the minimum curve radius, based on speed, friction and superelevation values.

$$Rmin = \frac{v^2}{127(e+ft)} \qquad \text{(Equation 2)}$$

Where: Rmin = Minimum Curve Radius (m)

V = Speed (Km/h)

e = Superelevation (m/m)

Ft = Coefficient of Transverse Friction

The minimum curve radii values used at the design stage range from about 100m for a design speed of 50 Km/h to about 500m for a design speed of 100 Km/h (Figure 6.4).

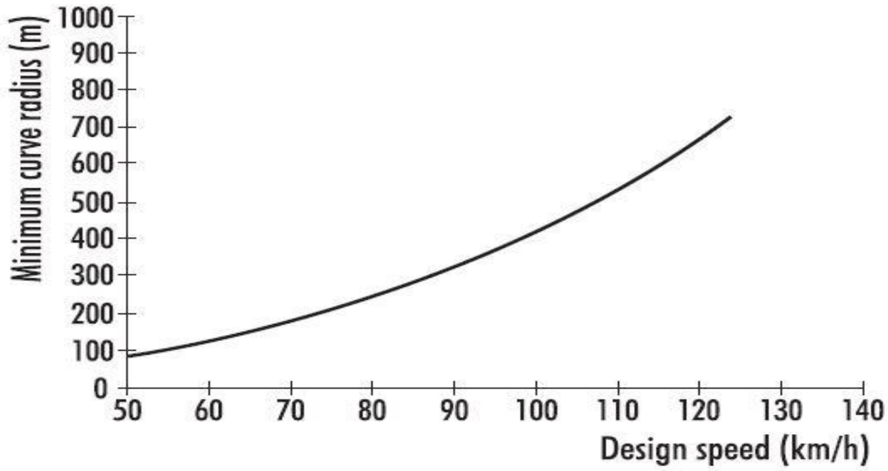


Figure 6.4: Minimum Curve Radius and Design Speed (Source: Kramme et Garnham, 1995)

Such radius values may be calculated with equation 2, using low coefficients of transverse friction, in order to:

-Take into account difficult drive conditions (wet pavement and worn tires).

-Avoid substantial increases in curve breaking distances.

-Provide vehicle occupants with an acceptable level of comfort.

●**Safety:** On rural roads, accident frequencies are generally seen to increase as curve radii decrease. A convex downward relationship is often found as shown if figure 6.5 below.

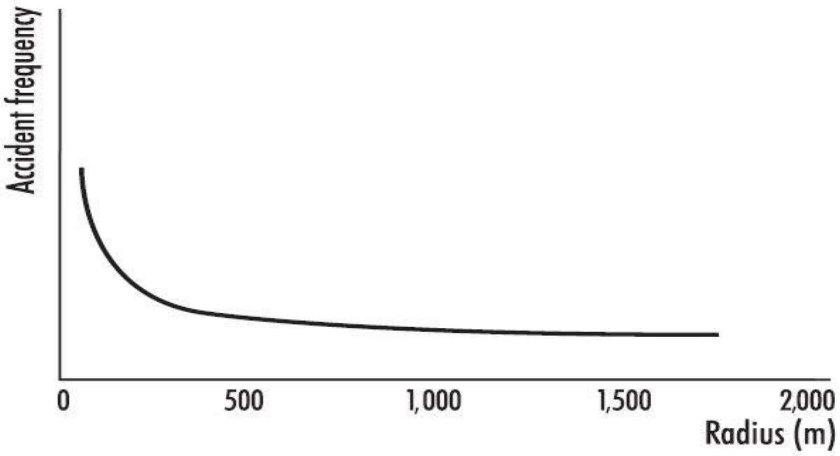


Figure 6.5: Accident frequency and curve radius

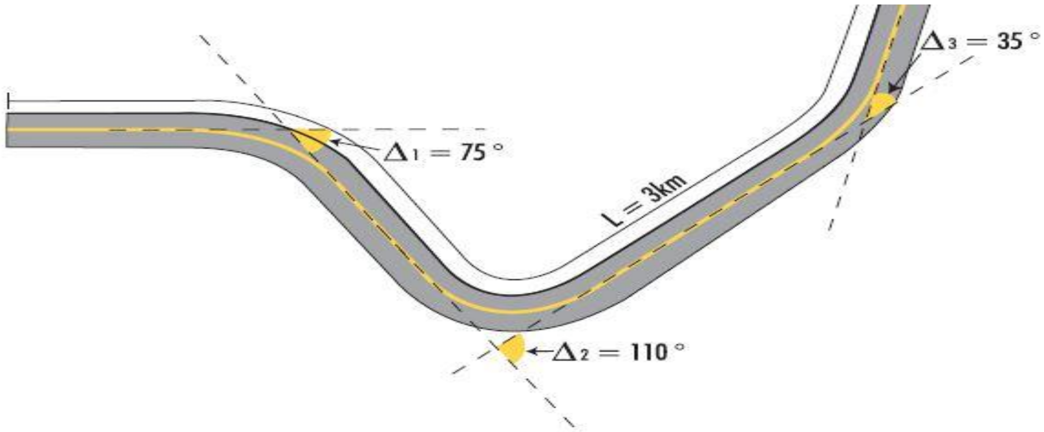
→The increase of accidents is significant when the radius is less than 400m.

However, the accident frequency in a curve is not influenced only by the characteristics of the curve itslef (radius, deflection angle, friction, superelevation, etc.) but also by the characteristics of the road alignment prior to the curve (length of tangent prior to the curve and general bendiness of the road).

It is, therefore, not surprising for two similar curves to have different safety performance depending on the road context in which they are located.

●**General Bendiness[1]:** The general bendiness of a road has a direct effect on the drivers level of attention and expectation with respect to the forthcoming road alignment. A sharp curve is more hazardous on a fairly straight road than a winding one. Figure 6.6 shows how to calculate Bendiness:

[1] Bendiness is defined as the sum of changes in direction (in degrees) per kilometer. Note: one gon = 0.9^{o}.



$$\text{Bendiness} = \frac{75° + 110° + 35°}{3\ \text{km}} = 73° / \text{km}$$

Figure 6.6: Road Bendiness

●**Irregular Curve Radius:** Marked changes in radius in a curve are to be avoided since they may surprise drivers and increase the risk of error. The accident risk is higher when a small curve follows a larger one. Yerpez and Fernandez (1986) found that 50% reduction of curve over a distance of more than 30m increases the number of accidents. An irregular radius can be converted into a uniform circular radius or clothoid or a combination of both without a major changes in road alignment.

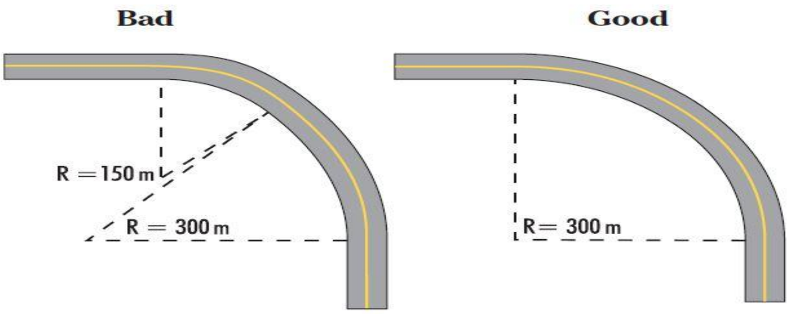


Figure 6.7: Irregular Curve Radius

●**Spiral Curve:** Spiral curves (also called transition curves or clothoids) are the third element in a horizontal alignment, along with tangent and circular curves. According to Lamm et al. (1999), spiral curves:

-Improve driving comfort by allowing a natural increase and decrease in centrifugal force as a vehicle enters and leaves a circular curve.

-Minimize encroachments and increase speed uniformity.

-Facilitate water runoffs in the superelevation transition zone.

-Enhance the appearances of the highway by eliminating noticeable breaks at the beginning and end of circular curves.

Spiral Curves are calculated using this formula:

$$\boldsymbol{R = \frac{A^2}{Ls}} \qquad \text{(Equation 3)}$$

Where: R = Curve Radius (at distance L) (m)

A = Parameter of the Spiral Curve (m)

Ls =Distance traveled from the starting point of the Curve (m)

Figure 13 shows the resulting curves for A= 150m and A= 300m

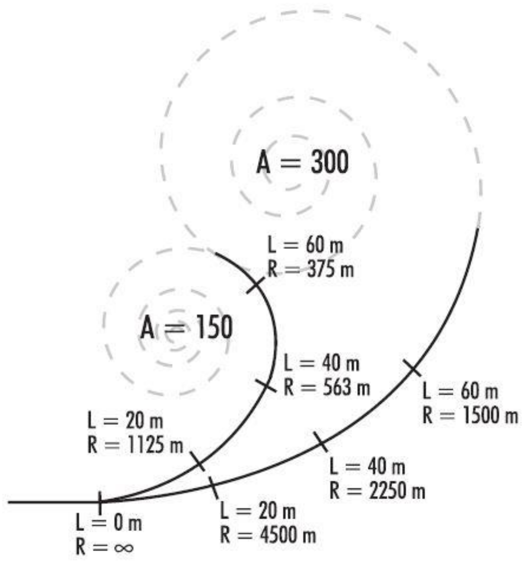


Figure 6.8: Spiral Curves

Overly long spiral curves should be avoided as they can hinder the visual perception of the curve and may contribute to drainage problems.

According to Council (1998), a spiral curve reduces accident rates by 8% to 25% on roads with high design standards. However, safety improvements brought about by transition curves are less evident on roads with lower geometric standards.

Other studies report contradictory results, which probably led Lamm et al. (1999) to conclude that:

« *Generally, speaking with respect to safety effects, the application of clothoids should not be overemphasized in the design process as it has been done so far in several countries. Of course, one should not forget the importance of other design impacts that transition curves provide, besides accident related issues …* »

6.1.3 Speed Differentials:

Operating speed is influenced by several factors related to the driver, road and road side conditions, vehicle characteristics, traffic conditions and weather conditions. Road alignment is undoubtedly the most important factor among road characteristics that influence driver's speed.

Speed variations along a road have direct impact on road safety. High standard roads should be designed to allow drivers to travel safely at a relatively constant speed that meets their needs and expectations. Otherwise driving errors are likely to occur.

In the early seventies, German researchers developed rules to help designers choose horizontal alignment sequences that would reduce operating speed variations along a route. The method known as Relation Design is seen as a major improvement over traditional design methods that merely checked compliance with minimum radii values.

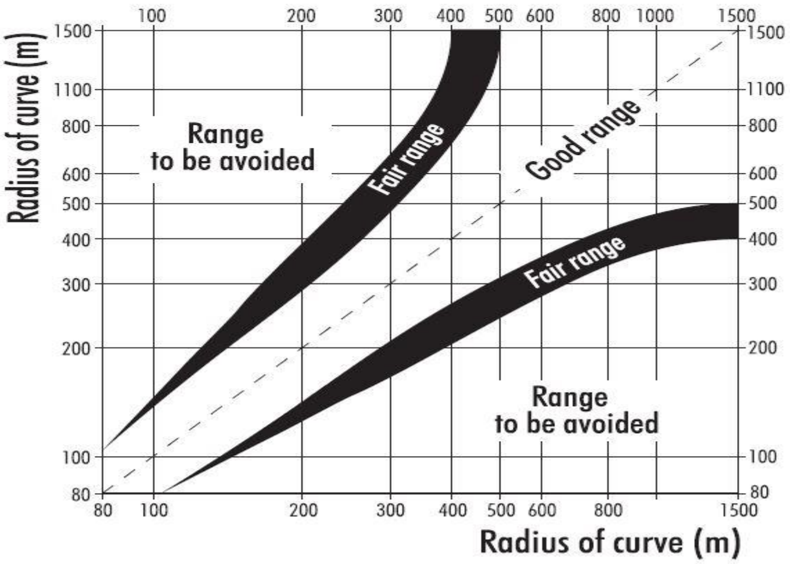


Figure 6.9: Tuning Radii in Cure Sequences (Source: German Design Guidelines, from Lamm et al.1999)

Relation design rules can also be expressed in terms of speed differentials. Lamm et al. (1999) recommend that a road's design quality can be assessed by comparing the 85th percentile speed of passenger cars (V85) on two successive road segments. If the differential is less than 10 km/h, the design is deemed good; between 10 km/h and 20 km/h, acceptable; over 20 km/h, poor. Spain uses similar criterion, but based on the 99th percentile speed (V99).

Table 6.1 below shows a small comparison between Lamm et al (1999) and Spain:

LAMM ET AL 1999		SPAIN	
SPEED DIFFÉRENTIAL ΔV_{85} (km/h)	DESIGN QUALITY	SPEED DIFFÉRENTIAL ΔV_{99} (km/h)	DESIGN QUALITY
< 10	Good	< 15	Good
10 - 20	Acceptable	15 - 30	Fair
> 20	Poor	30 - 45	Poor
		> 45	Dangerous

Table 6.1: Design Quality – Speed Differentials (Source: Lamm et al. in highway design and traffic safety engineering handbook, 1999)

●**Safety:** Anderson et al. (1999) analyzed the impact of operating speed differentials (V85) on accidents. Based on data collected at 5287 curves, that found that accident rate in curves with speed differential of over 20 km/h is two times higher than in those with a speed differential of 10 km/h to 20 km/h and six times higher than in those with a speed differential of less than 10 km/h (Figure 6.10).

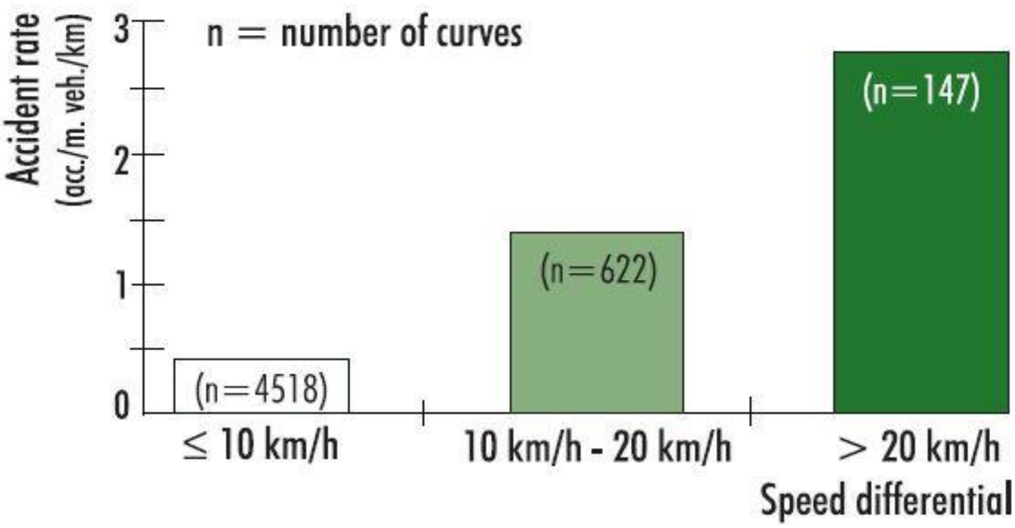


Figure 6.10: Accident rates and Speed differentials (Source: Anderson et al. (1999))

6.1.4 Surface Condition:

The available transverse friction in a curve has a strong impact on the maximum speed on which it can be driven. For example, for a curve with a radius of 300m, a superelevation of 0.06 and a coefficient of transverse friction (ft) of 0.30, the maximum (theoretical) speed is 108 km/h. With aft of 0.80 it raises to 148 km/h. Transverse friction values used at the design stage (ftd) are generally much lower than transverse friction values that are available on roads.

The choice of ftd values is based on the following objectives:

-Providing a safety margin for adverse weather conditions.

-Avoiding excessive increase in breaking distances in curves (Figure 6.11).

-Offering a comfortable ride to vehicles occupants.

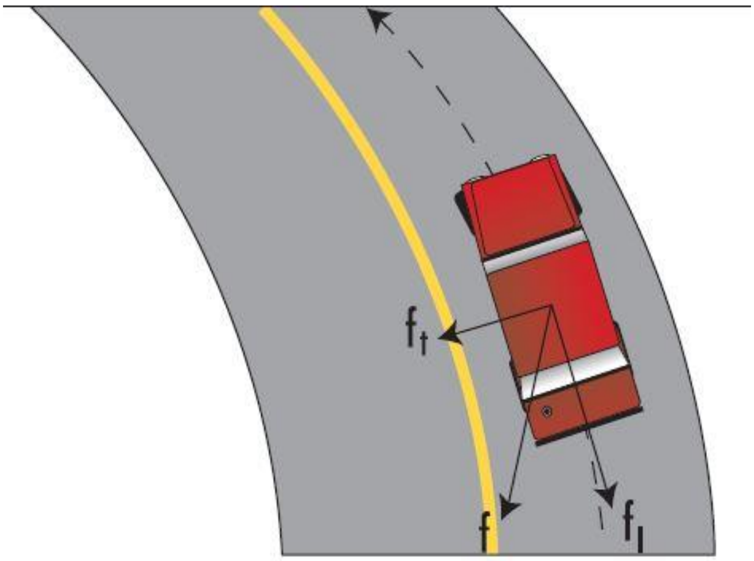


Figure 6.11: Friction in Horizontal Curve

$$f^2 = fl^2 + ft^2 \qquad \text{(Equation 4)}$$

Where: f =Coefficient of Friction (total)

fl =Coefficient of Longitudinal Friction

Ft =Coefficient of Transverse Friction

→When braking in a curve, the total available friction is distributed between its longitudinal component (fl), required for braking, and its transverse component (ft), required for changes in direction. To avoid excessive increases in braking distances, ftd values not greater than 40% to 50% of the total friction expected under difficult conditions are selected, so that around 90% of the total friction remains available for braking manoeuvres.

As a result, horizontal curves can often be driven faster than design and posted speeds (under favorable conditions). Realizing this, a number of drivers adopt relatively high speeds, thereby reducing their safety margin. It is a driving habit that may prove to be hazardous if the skid resistance at a specific curve low and the driver does not decelerate sufficiently. When the friction available in the curve becomes lower than the required friction, the driver loses the control of his vehicle. The required friction (fr) can be calculated with the following equation:

$$fr = \frac{V^2 85}{127R} - e \qquad \text{(Equation 5)}$$

Where: R= Curve Radius

V85= Speed (km/h)

e= Superelevation (m/m)

fr= Friction Required at V85

Lamm et al. (1999) recommend assessing the quality of a horizontal alignment by comparing the coefficient of transverse friction used in design (ftd) with the coefficient of friction required (fr) to take the curve. (Table 6.2)

FRICTION DIFFERENTIAL	DESIGN QUALITY
$f_{td} - f_r \geq +0.01$	Good
$-0.04 \leq f_{td} - f_r < +0.01$	Acceptable
$f_{td} - f_r < -0.04$	Poor

Table 6.2: Design Quality-Friction differentials (Source: Lamm et al. in highway design and traffic safety engineering handbook, 1999)

fr= Friction Coefficient Required at speed V

ftd =Coefficient of Transverse Friction (Design)

●**Safety:** The presence of water between a tire and a road surface reduces the available friction. As a result, concentration of wet surface accidents may be indicative of a friction deficiency. The problem is more important with horizontal curves where friction requirements are higher.

●**Skidding:** Skidding occurs when the centrifugal force becomes greater than the resistance provided by the transverse friction (ft) and the superelevation (e). In such conditions, the driver loses control of his vehicle, which is pushed toward the outside of the curve.

The speed at which the Skidding may occur should always be significantly higher than the posted speed. Otherwise, appropriate warning devices need to be installed sufficiently ahead of the curve to prepare divers to the forthcoming situation.

$$\boldsymbol{Vskid} = \sqrt{\mathbf{127}\boldsymbol{R}}(\boldsymbol{e} + \boldsymbol{ft}) \qquad \text{(Equation 6)}$$

Where: Vskid= Skidding speed (km/h)

R= Radius of the curve (m)

e= Superelevation (m/m)

f= Coefficient of transverse Friction

●**Jack-knifing:** For various reasons, skidding may not be reached simultaneously on all wheels of a vehicle: different wheel loads, braking forces on each wheel, tire characteristics, road surface characteristics, etc.

If a vehicle has a rigid configuration (e.g. single unit truck), friction can still develop at wheels which have not yet reached the skidding threshold. In the case of articulated vehicles (semi-trailers, trailers...), the sliding of some wheels may initiate the rotation of its various rigid components around its kingpins and change the overall configuration of the vehicle. This process is called jack-knifing, which is more likely to occur on wet pavement or during braking manoeuvres. Where semi-trailers account for a significant proportion of total traffic, jack-knifing may occur before skidding.

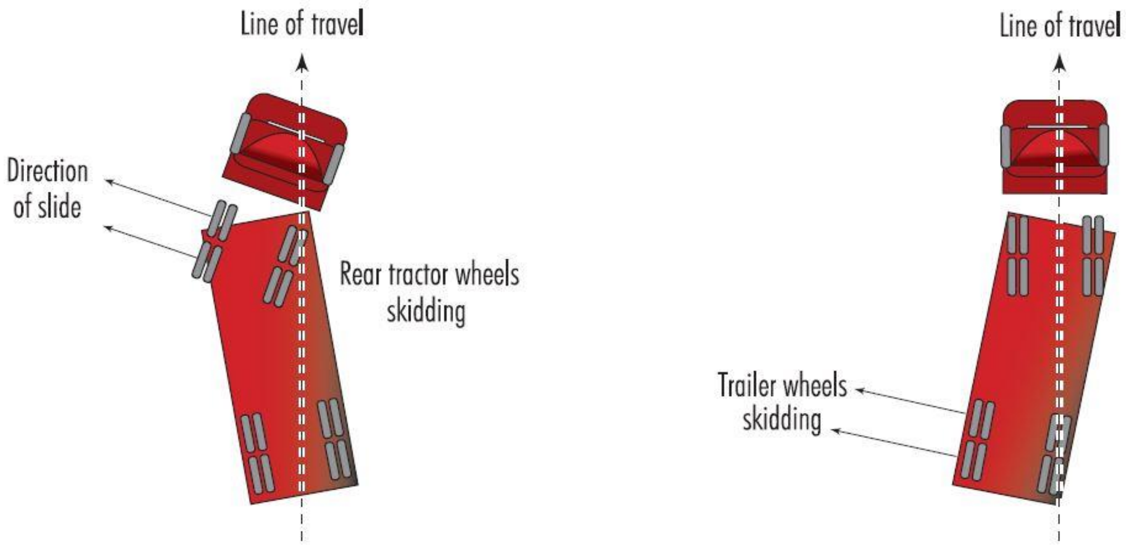


Figure 6.12: Jack-knifing

6.1.5 Overturning:

When the available friction is high, some heavy vehicles may overturn before skidding. A vehicle's overturning threshold (OT)[2] or Static Stability Factor (SSF) depends on its truck width and the height of its center of gravity:

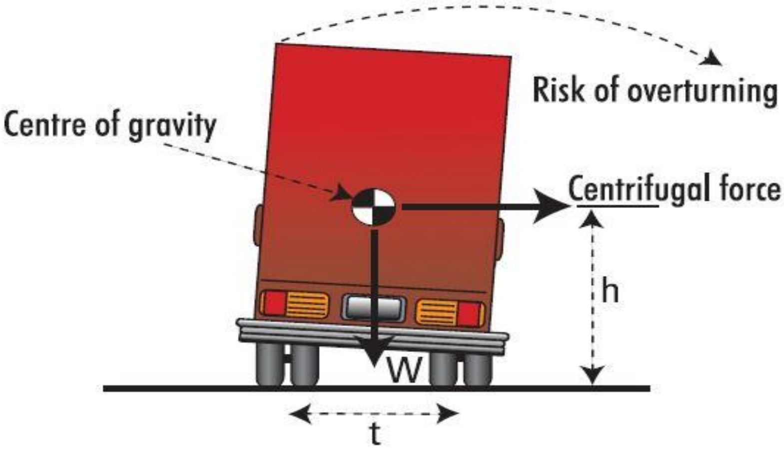


Figure 6.13: Overturning

OT= t/2h (Equation 7)

Where: OT: Overturning threshold

T: Track Width (m)

H: Height of the center of gravity (m)

When the OT value is larger than the coefficient of transverse friction that can be mobilized in a curve (ft), the vehicle will overturn before skidding, and inversely. The risk of overturning is usually low for passenger vehicles since their overturning threshold is relatively high (typically between 1 and 1.5g). However, some heavy vehicles have lower OT values (in order of 0.3 or 0.4) and their risk of overturning is much higher.

The overturning speed equation is similar to the skidding speed equation, except that the available coefficient of transverse friction (ft) is replaced by the overturning threshold (t/2h):

$$\boldsymbol{Vr} = \sqrt{}\mathbf{127}\boldsymbol{R}(\boldsymbol{e} + \frac{\boldsymbol{t}}{\mathbf{2}\boldsymbol{h}})) \qquad \text{(Equation 8)}$$

Where: Vr: Overturning Speed (km/h)

R: Radius of the Curve (m), and e: Superelevation (m/m)

[2] The overturning threshold value represents the lower value of centrifugal acceleration that is sufficient to cause a vehicle to rollover (it is expressed in terms of 'g').

6.1.6 Superelevation:

Superelevation is a road's transverse incline toward the inside of a horizontal curve (Figure 6.14). It slightly reduces the friction needed to counter the centrifugal force and increases riding comfort. As a result, the maximum speed in a curve increases with superelevation (Table 6.3).

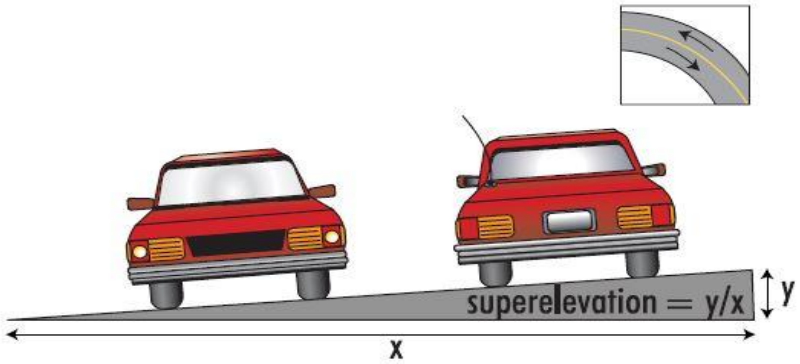


Figure 6.14: Superelevation in Curve

SUPERELEVATION (m/m)	SPEED (km/h)
0.00	62
0.02	67
0.04	71
0.06	76
0.08	80

Radius = 250 m, coefficient of friction = 0.12

Table 6.3: Example showing the Relationship between the Superelevation and Speed

→ Excessive superelevation may cause slow vehicles to slide toward the inside of the curve when the friction level is so low (icy conditions). Superelevation values ranging from 5% to 8% are recommended in design.

A transition zone between the tangent and the horizontal zone is needed to gradually introduce the superelevation. In part of this zone, the road profile becomes flat on its outer side, which can lead to water accumulation and contribute to skidding (Figure 6.15).

The end of this flat zone must be located before the start of the curve and special attention must be paid to the quality of drainage in that area.

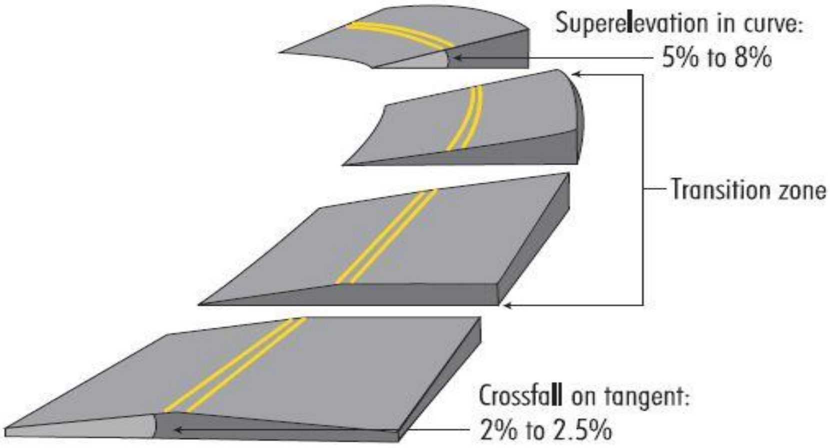


Figure 6.15: Superelevation Development

●**Safety:** Dunlap et al. (1978), found the number of accidents on wet pavements to be abnormally high in curves with a superelevation of less than 2%.

Zegeer et al. (1992), report that improving the superelevation reduces the number of accidents by 5 to 10%.

6.1.7 Road Width:

In horizontal curves, the radius followed by a vehicle's front wheels is larger than the radius of its rear wheels, which increases the width swept (as compared to the situation in a tangent) (Figure 6.16). The additional width is negligible in the case of passenger's vehicles but can be significant with long articulated vehicles. Moreover, the difficulty stemming from changes in direction in a curve increases the risk of encroachment outside the traffic lane.

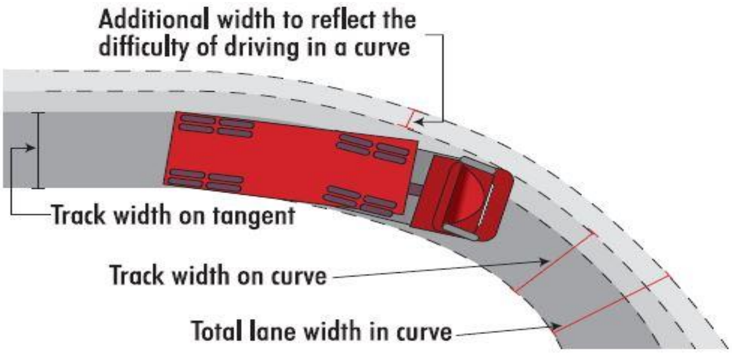


Figure 6.16: Lane Widening in a Curve

→ As a result, road width often needs to be increased in horizontal curves. The required width depends on the curve radius, operation speed and vehicle's characteristics.

« *Traffic volumes should also be considered. For example the Canadian design guide indicates that on two-lane roads, pavement widening is not required when the there is less than 15 trucks/h in both directions*» (Transportation Association of Canada, 1999).

●**Safety:** Increasing road width reduces accident rates (Figure 6.17).

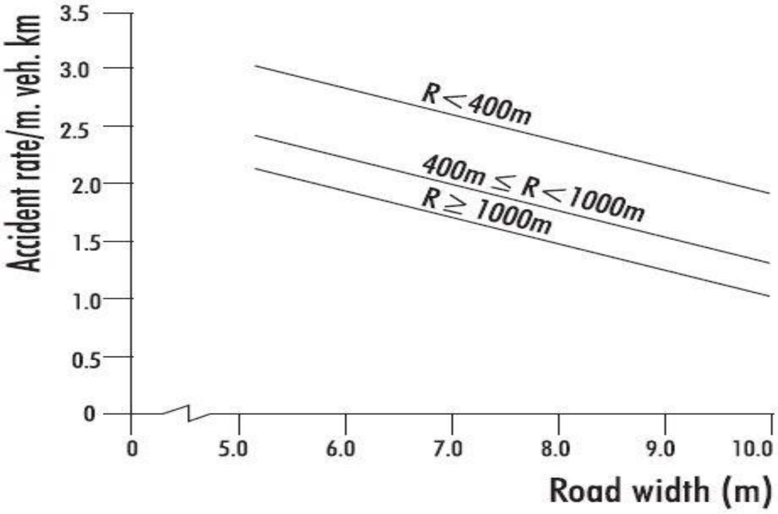


Figure 6.17: Accident rate in curve and road width (Source: Krebs and Kloeckner (1977))

Table 6.4, based on a USA study conducted by Zegeer et al. (1990), shows accident reductions that can be expected from traffic lane/shoulder widening in curves.

WIDENING (m)		ACCIDENT REDUCTION (%) WIDENING OF		
TOTAL	PER SIDE	LANES	PAVED SHOULDERS	UNPAVED SHOULDERS
0.6	0.3	5	4	3
1.2	0.6	12	8	7
1.8	0.9	17	12	10
2.4	1.2	21	15	13
3.0	1.5		19	16
3.6	1.8		21	18
4.2	2.1		25	21
4.8	2.4		28	24
5.4	2.7		31	26
6.0	3.0		33	29

Table 6.4: Accident Reduction (%) Due to Lane or Shoulder Widening (Source: Zegeer et al.1990)

6.1.8 Road Sides-Sight Distance:

As anywhere else on the road, the sight distance at any point of a horizontal curve must be sufficient to allow safe stopping manoeuvres. Various obstacles located on the inside of curves can hinder visibility, such as embankments, vegetation, buildings, and so on. Adequate Lateral Clearance (LC) on the inside of curves must be provided to ensure safety.

The size of the lateral clearance zone depends on the curve braking distance (Figure 6.18).

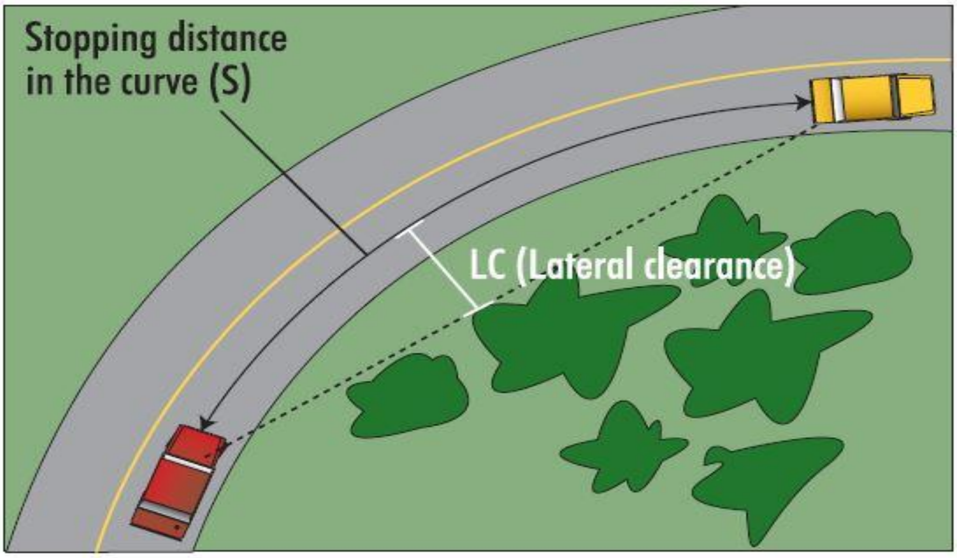


Figure 6.18: Lateral Clearance in Curve

The braking distance for heavy vehicles equipped with conventional braking systems is significantly longer than for passenger vehicles (Table 6.5). In some instances, the higher eye-level compensates for the increased braking distance. However, this may not be the case when road side objects are high. In such situations, the lateral clearance distance should be calculated for heavy vehicles.

Table HA-7 Stopping distances – Passenger cars and trucks

	DESIGN SPEED (km/h)							
	40	50	60	70	80	90	100	110
Stopping distances (m)								
- Passenger cars	45	65	85	110	140	170	210	250
- Trucks	70	110	130	180	210	265	330	360

Table 6.5: Stopping Distances-Passenger Cars and Trucks (Source: Transportation Association of Canada 1999).

6.1.9 Road Sides-Forgiving Road:

The roadside encroachment rate is much higher in curves than in tangents. According to the roadside design guide (American Association of State Highway and Transportation Officials, 2002), this rate is as much as four times higher on the outside of curves as in tangents and as much as twice as high as on the inside of curves (Figure 6.19).

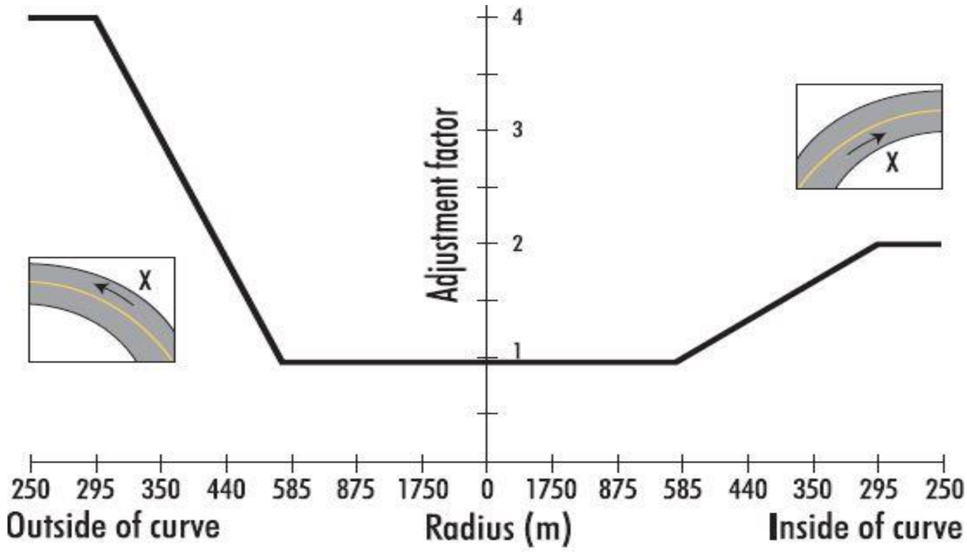


Figure 6.19: Encroachment Factors (Source: Roadside Design Guide, 2002, by the American association of state highway and transportation officials, Washington, D.C. Used by Permission)

Steep side slopes are simply another kind of roadside obstacles and should be avoided. The maximum gradient that can be travelled by errant vehicles is in the order of 1:3 to 1:4. The angle between shoulder/slope and slope/adjacent land should also be smoothed.

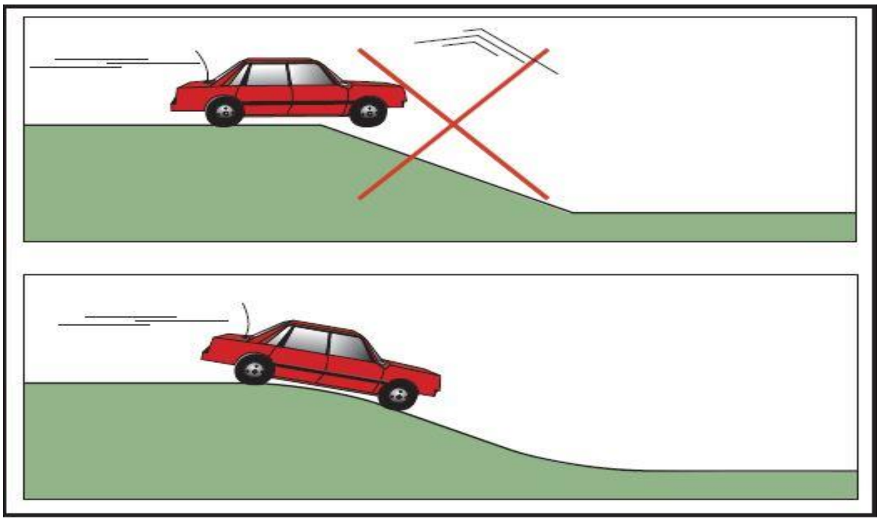


Figure 6.20: Smoothing of Side Slopes

6.1.10 Passing:

Passing opportunities should be considered in the curve itself and in a longer road segment comprising the curve and several kilometers of roads around it.

●In the Curve:

Straight segments or curves with long radii are required to obtain the necessary sight distance to pass safely. Passing is seldom possible in a right-hand curve unless it has a very long radius, and even then, it is dubious manoeuvre that should be avoided since the passed vehicle impedes visibility (right-hand side driving). Whenever the sight distance is not sufficient, pavement marking should clearly (and at all times) prohibit passing.

●On a Longer Road Segment:

On two-lane rural roads, there must be sufficient passing opportunities to avoid long vehicle queues and dangerous manoeuvres that may result[3]. Some coutires recommend minimum alignment percentages with passing sight distance (Table 6.6).

COUNTRY	MINIMUM PERCENTAGE
SWITZERLAND, GERMANY	20%
FRANCE	25%
GREAT BRITAIN	15 - 40% (depending on the road category)

Table 6.6: Minimum percentage of Alignment with passing sight distance (Source: Road Safety Manual 2003)

[3] Sight Distance can be restricted not only by the presence of horizontal curves but also by features of the vertical alignment and by combination of both.

6.2 Vertical Alignment:

6.2.1 General Principles:

The Vertical Alignment for a road consists of straight segments (Leveled or Inclined) connected by sag or crest curves. Combinations of these elements create various shapes of road profiles (Figure 6.21).

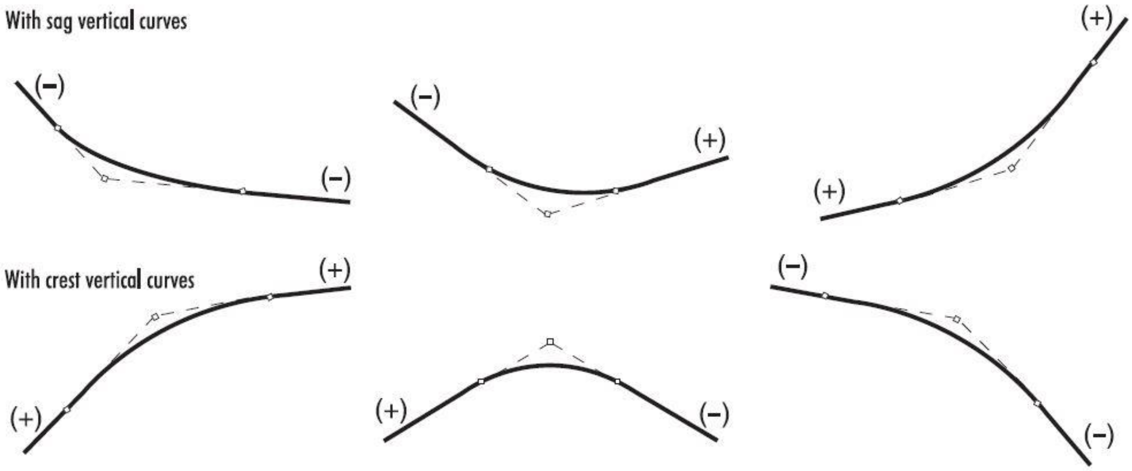


Figure 6.21: Examples of Vertical Alignments

The Grade percent (G), the height of slope (y) and K value characterize the vertical alignment (Figure 6.22).

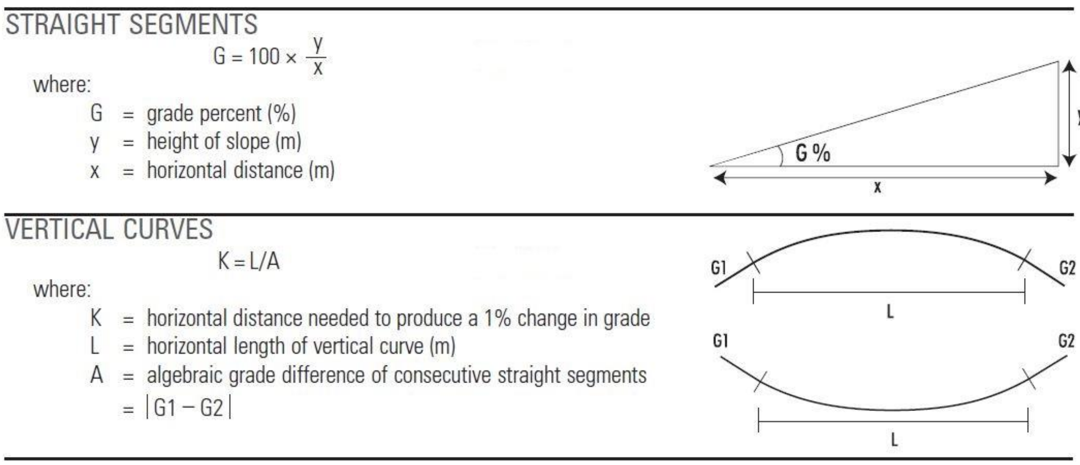


Figure 6.22: G, y and K values

6.2.2 Downhill Grades:

The main problem in downhill grades is related to heavy vehicles. In fact, at such locations, the main elements that need to be considered are the increase in stopping distances and the possibility of overheating (Especially for heavy vehicles).

6.2.2.1 Brake Temperature:

Generally, the brake temperature depends on the following factors:

*Downhill Speed

*Vehicle mass

*Initial brake temperature

*Percentage and length of grade

*Emergency Stop in descent

Usually, the critical brake temperature is around 260°C. Above this point, the efficiency of braking systems is reduced due to various physical phenomenons like expansion and deformation etc.

6.2.2.2 Stopping Distance:

In such conditions, the increase in stopping distance is noticeable. Otherwise, the stopping distance in downhill grades is significantly greater than that in a levelled road.

DOWNHILL GRADE (%)	BRAKING DISTANCE (m)
0	210
5	241
10	288

Table 6.7: Braking Distances in downhill grades

With an initial speed of 100 km/h and a friction coefficient of 0.28, as shown in the results in the table 6.7 above, the braking distance increases by 37% (78 m) when comparing a 10% grade to a levelled road.

6.2.2.3 Road Signs at Downhill Grades:

As well known, covering the entire road network, drivers should have sufficient information about the road's profile. However, in such locations (Downhill grades), the situation becomes more critical; Thus, drivers (especially those with heavy vehicles) should have enough information about the grade's profile before starting their descent to be able to adjust their speed right from the top, and by consequence, to avoid difficult deceleration maneuvers midway down. The figure 6.23 below illustrates the foregoing. It indicates both percentage and length of grade.

Figure 6.23: Downhill warning Sign

'*Signs must not be located too far from the beginning of the slope as to diminish their credibility. The recommended distances should be adapted to the operating speed. Warning signs should be located from 25 m (30 km/h) to 200 m (100 km/h), before the beginning of the descent'* (Baass, 1993).

6.2.2.4 Drainage:

Generally, water flows on grades may accelerate the road's deterioration. Thus, drainage facilities should allow the rapid clearance of water and prevent the accelerated erosion of the road surface. Moreover, Drainage capacity has to be suited to the rainfalls quantity reasonably expected in the road's area, also, it has to be regularly maintained to prevent clogging from accumulated debris.

It is not recommended to put deep and open drainage structures near the roadway, because they constitute rigid obstacles that may aggravate accident severity.

6.2.2.5 Brake Check Areas:

Built at the top of some long and steep grades, brake check areas give heavy vehicle drivers the opportunity to stop and check the condition of their vehicle braking system away from traffic. In addition to forcing drivers to start their downhill run from a stopped position, brake check areas provide heavy vehicle drivers with more detailed information on the configuration of the grade (Figure 6.24).

Figure 6.24: Brake Check Area

6.2.2.6 Arrester Beds:

Arrester beds are roadside facilities designed to stop runaway heavy vehicles. Granular material is used (5-10 mm, rounded), as it provides more rolling resistance and reduces the length of the bed.

Depending on the terrain, the arrester bed can be either: Horizontal, ascending or descending (Figure 6.25). Ascending designs reduce stopping distances but it makes the use of granular material mandatory, to prevent rollback onto the roadway.

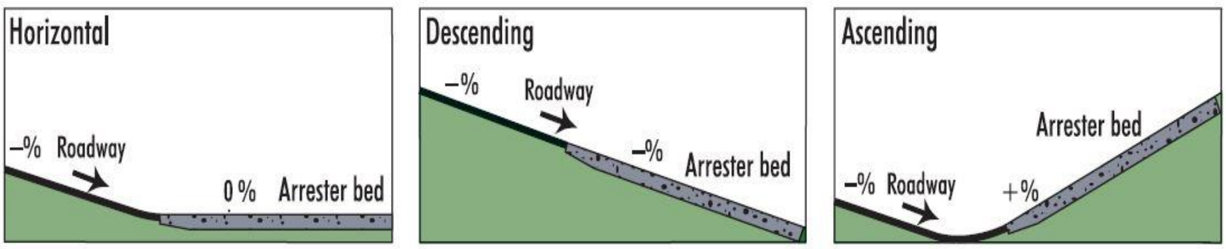


Figure 6.25: Types of Arrester Beds

Arrester Beds should preferably be located on a tangent section since their location on curves will add to the maneuvering difficulties facing the driver of a runaway truck.

Advance warning signs and distinctive markings should be used to identify the presence of the arrester bed and guide drivers of out-of-control vehicles.

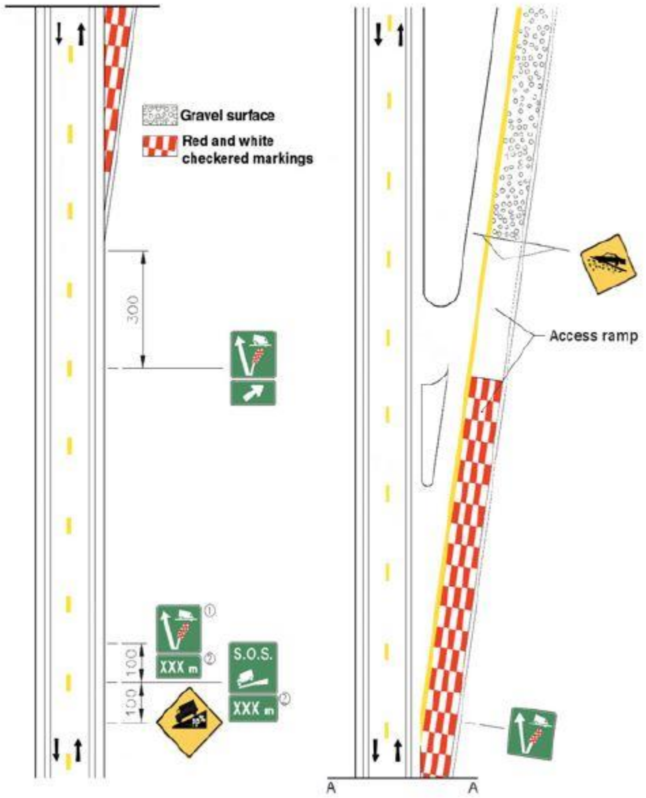


Figure 6.26: Example of Arrester Bed (Source: Ministère des Transports du Quèbec, 1999)

→Due to their high construction costs, arrester beds are built only on a limited number of grades with a history of truck accidents, after the failure of other less costly measures.

6.2.3 Uphill Grades:

6.2.3.1 Generalities:

The maximum speed for a vehicle on an uphill grade depends on its mass/power ratio. For passenger cars, this ratio is sufficiently small to maintain a constant speed on most uphill grades. However the much higher mass/power ratio of heavy vehicles may cause significant slowdowns on uphill grades.

Ratios of 180Kg/KW or 8.0hp/ton are usually used at the design stage to estimate heavy vehicles accelerations and decelerations on grades. An example of deceleration curves is shown in figure 6.27. It indicates that even on a 1% uphill grade, a truck is slowed down. The deceleration rate and the speed reduction rapidly increase with the steepness of the grade.

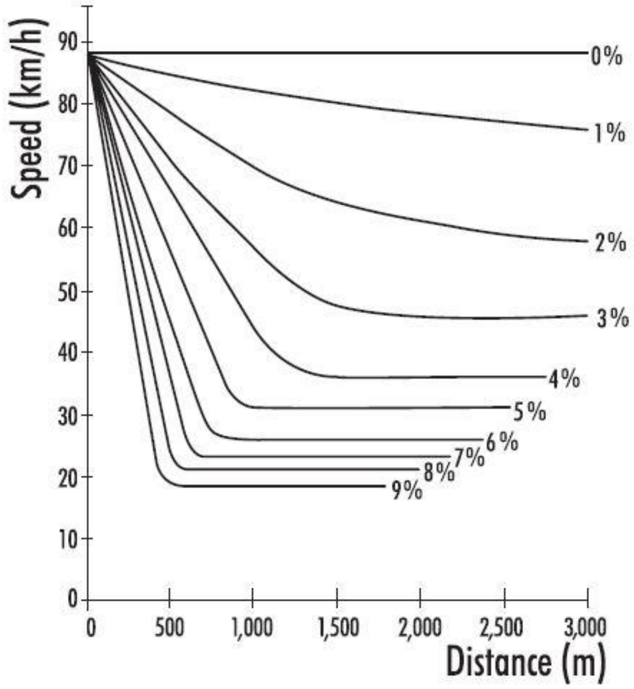


Figure 6.27: Deceleration Curves (Source: A policy on the geometric design of highways and streets, 1994, by the American association of state highway and transportation officials, Washington, D.C. Used by permission)

6.2.3.2 Climbing Lanes:

Auxiliary lanes can be constructed on uphill grades to enable safe passing manoeuvres by slower vehicles. The criteria used to justify the construction of an auxiliary lane differ from one country to another; they are based on a comparison of a heavy vehicle climbing speed with either: -An absolute minimum speed.

-The heavy vehicle's speed prior to uphill run.

-A passenger car's climbing speed.

In addition to the speed differentials, traffic volumes (total and heavy vehicles) are often considered.

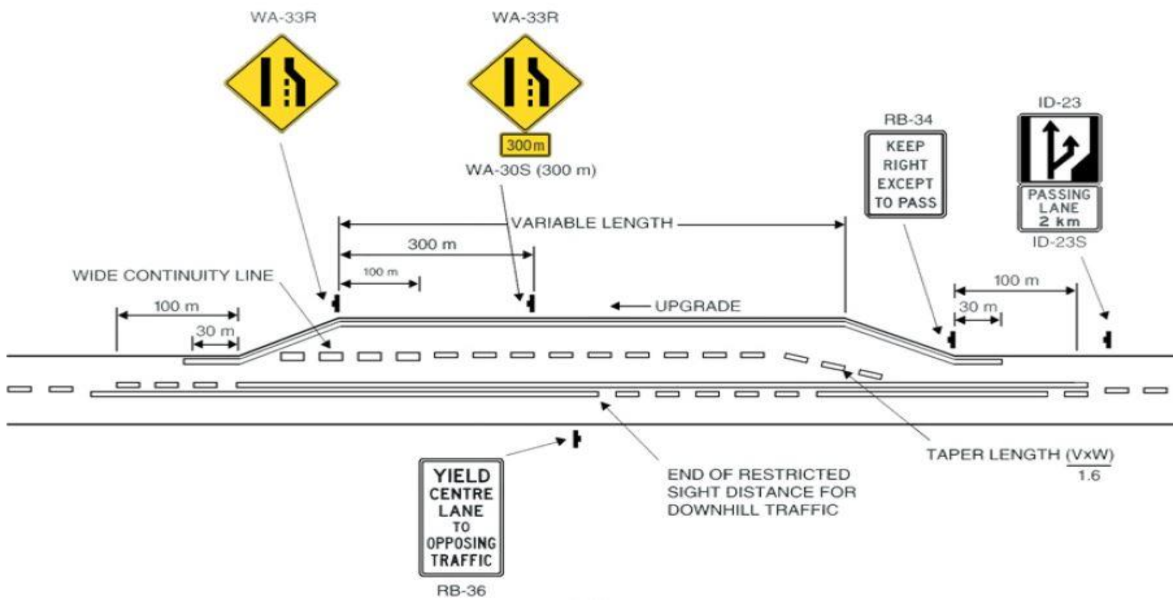


Figure 6.28: Climbing Lane (Source: Transportation Association of Canada, 1999)

→We have to note that, auxiliary lanes encourage passing manoeuvres at relatively high speed that are incompatible with the slower speeds of vehicles accessing and exiting the road. Therefore, auxiliary lanes should not be located in conjunctions with intersections or other access points.

Figure 6.29: Combination of intersection and climbing lane at uphill grade to be avoided in the design

●Climbing Lane- Traffic Management:

There are two different ways to manage traffic flow at climbing lanes:

-Main traffic on outside lanes (The inside lane is reserved for passing)

-Main traffic on inside lanes (The outside lane is reserved for slower vehicles)

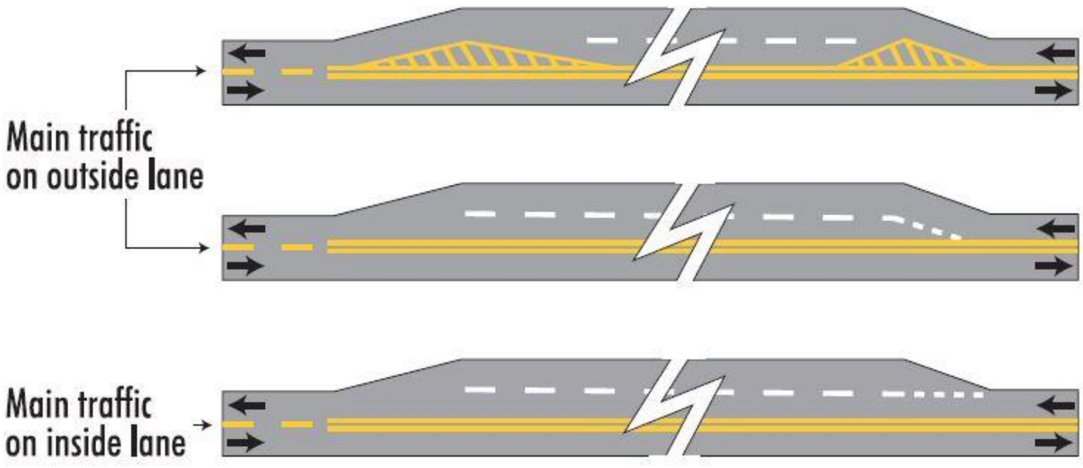


Figure 6.30: Climbing Lane and Traffic Management

The first method is preferred as it is consistent with traffic management rules on the rest of the road network (Vehicles stay on the outside lane except when passing) and it makes the climbing lane more effective.

6.2.4 Vertical Curves:

6.2.4.1 Generalities:

As previously mentioned, the straight segments of a vertical alignment (leveled or inclined) are connected by sag and crest curves. These curves are characterized by their K value[4]. As K decreases, the curve becomes sharper which reduces the available sight distance.

Figure 6.31: Examples of K value

Sight distance problems are more frequent on crest curves than on sag curves, where one should nevertheless verify that the visibility is not reduced by either the angle of vehicle's headlight beam (at night) or the presence of overhead structures (Viaduct, road signs, etc.). As anywhere else on the road network, the available sight distance should always be, at vertical crests, equal or larger than the required stopping distance.

[4] K = 100 * R (Vertical Curve Radius)

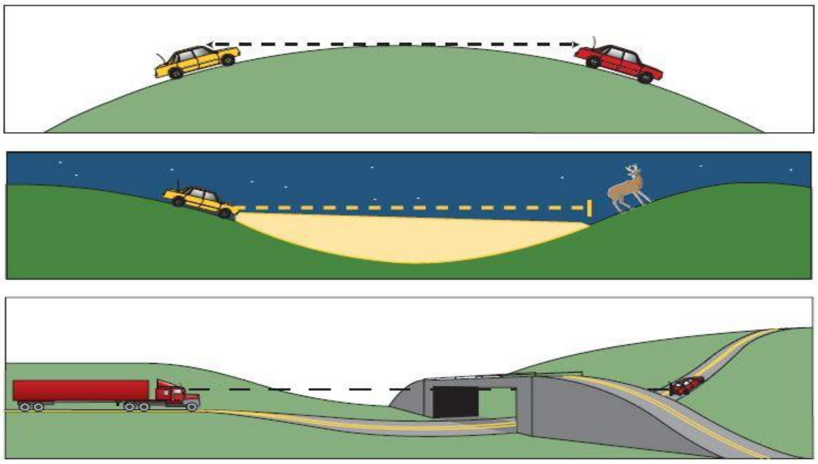


Figure 6.32: Sight Distance Restrictions on Sag and Crest Curves

6.3 Sight Distance:

6.3.1 General Principles:

At any point on a road, the available sight distance must be sufficient for a driver travelling at a reasonable speed (V85) to stop this vehicle safely without hitting a stationary object on his path. At intersections, other sight distance criteria need to be satisfied to ensure driver's safety. These criteria vary according to the type of intersection, the right-of-way rules and the allowed maneouvres.

TYPE OF INTERSECTION AND RIGHT-OF-WAY RULES	SIGHT DISTANCE CRITERIA			
	STOPPING	MANOEUVRE	TRIANGLE	DECISION
		Crossing from minor Turning from minor Turning from major		
Conventional intersections				
Uncontrolled	X	X	X	Complex or unexpected situations
Yield	X	X	X	
Stops on minor road	X	X		
All-way stop	X			
Traffic signals	X	X		
Roundabouts	X		X	

Table 6.8: Sight Distance Criteria at Intersections

→*«The relationship between sight distance and accident rate is not linear since the rate is seen to increase rapidly after a certain critical distance. »* (Fambro et al. 1997)

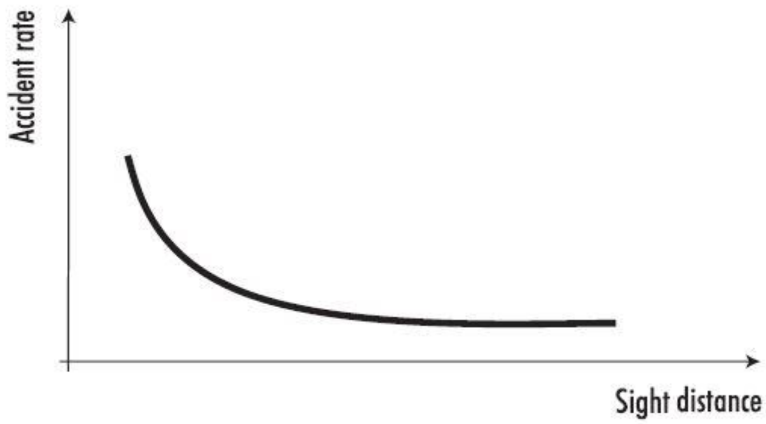


Figure 6.33: Sight Distance and Accident Rate

→On rural roads, the critical sight distance is in the order of **90 m** to **100 m**.

6.3.2 Intersections:

6.3.2.1 Stopping Sight Distance:

As anywhere else in the network, the available sight distance[5] when approaching an intersection must be sufficient to enable a driver travelling at a reasonable speed (V85) to stop his vehicle safely.

At conventional intersections, the stopping sight distance should be checked on each of the intersection's upstream and downstream approaches. At a roundabout, the stopping sight distance should be checked on each approach, in the ring lane and in each of the roundabout's exit (Figure 6.34). Specific attention should be paid to the sight distance of pedestrian crosswalks in the roundabout exits.

[5] When measuring available sight distance, lines of sight should not encroach on roadside areas that are not permanently free of visual obstruction. One needs to take into account temporary or seasonal obstructions that may be not present during the site visit (Vegetation, Snow, etc.)

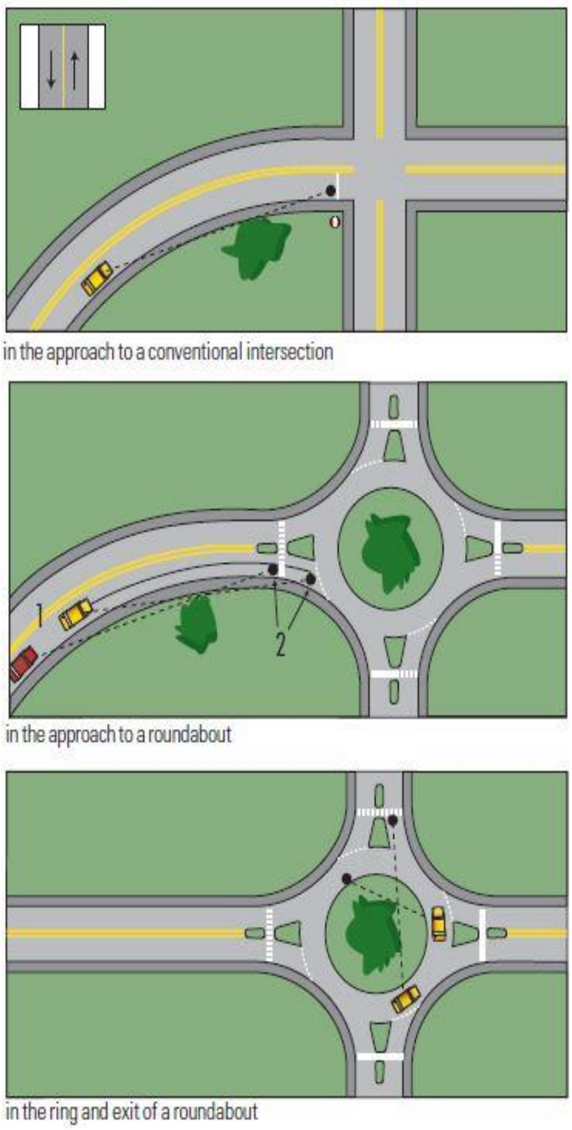


Figure 6.34: Stopping Sight Distance

The two following equations, which use either the coefficient of longitudinal friction or the deceleration rate, may be used to calculate the required stopping sight distance. The first term of these equations represents the distance travelled by a vehicle during a vehicle's reaction time whereas the second term represents the distance travelled during the mechanical braking of the vehicle (Typical values for each parameter of these equations are shown in the table 6.9).

$$\boldsymbol{SSD = Vi * \frac{t}{3.6} \frac{Vi^2}{254\left(fl \pm \frac{G}{100}\right)}} \qquad \text{(Equation 9)}$$

$$\boldsymbol{SSD = Vi * \frac{t}{3.6} \frac{Vi^2}{254\left(\frac{a}{g} \pm \frac{G}{100}\right)}} \qquad \text{(Equation 10)}$$

Where:

SSD: (required) stopping sight distance (m)

T: Reaction time (s)

Vi: Initial Speed (Km/h)

fl: Coefficient of longitudinal friction

a: Deceleration Rate (m/s²)

g: Acceleration due to Gravity (9.8 m/s²)

G: Grade percent (%)

PARAMETER	TYPICAL VALUES
Reaction time (t)[a]	1.0 to 2.5 s
Coefficient of longitudinal friction (f_l)[b]	0.15 to 0.5
Deceleration rate (a)	3.4 m/s²

Table 6.9: Typical Values of for Stopping Sight Distance Calculation

COUNTRY	TIME	SPEED (km/h)											
	(s)	30	40	50	60	70	80	90	100	110	120	130	140
		STOPPING SIGHT DISTANCE (m)											
Austria	2.0	-	35	50	70	90	120	-	185	-	275	-	380
Canada	2.5	-	45	65	85	110	140	170	210	250	290	330	-
France	2.0	25	35	50	65	85	105	130	160	-	-	-	-
Germany	2.0	-	-	65	85	110	140	170	210	255	-	-	
Great Britain	2.0	-	-	70	90	120	-	-	215	-	295	-	-
Greece	2.0	-	-	-	65	85	110	140	170	205	245	-	-
South Africa	2.5	-	50	65	80	95	115	135	155	180	210	-	-
Sweden	2.0	35	-	70	-	165	-	-	-	195	-	-	-
Switzerland	2.0	35	-	50	70	95	120	150	195	230	280	-	-
USA	2.5	35	50	65	85	105	130	160	185	220	250	285	-

Table 6.10: Recommended stopping sight distance in several countries (Adapted from: Harwood et al.1995)

6.3.2.2 Maneuvering Sight Distance:

A driver who is stopped at an intersection should have sufficient sight distance to complete safely all permitted but non-priority manoeuvres:

-Left turn, crossing, right turn from a minor road;

-Left turn from a major road

A number of methods that vary greatly in their complexity have been developed to calculate the required maneuvering sight distance. A simple equation calculates this distance based on the speed of vehicles with the right of way and the gap required completing non-priority manoeuvres. This gap may vary from a country to another, as shown in table 6.11.

$$\boldsymbol{D = V85 * \frac{t}{3.6}} \qquad \text{(Equation 11)}$$

Where: D = Maneuvering distance (m)

V85 = 85th percentile of speed on major road (Km/h)

T = Maneuvering gap

FRANCE	ENGLAND	SPAIN	USA
6 - 8 s	5 - 8 s	6 - 8 s	6.5 - 7.5 s

Table 6.11: Maneuvering gaps at intersection (Passenger Vehicle)

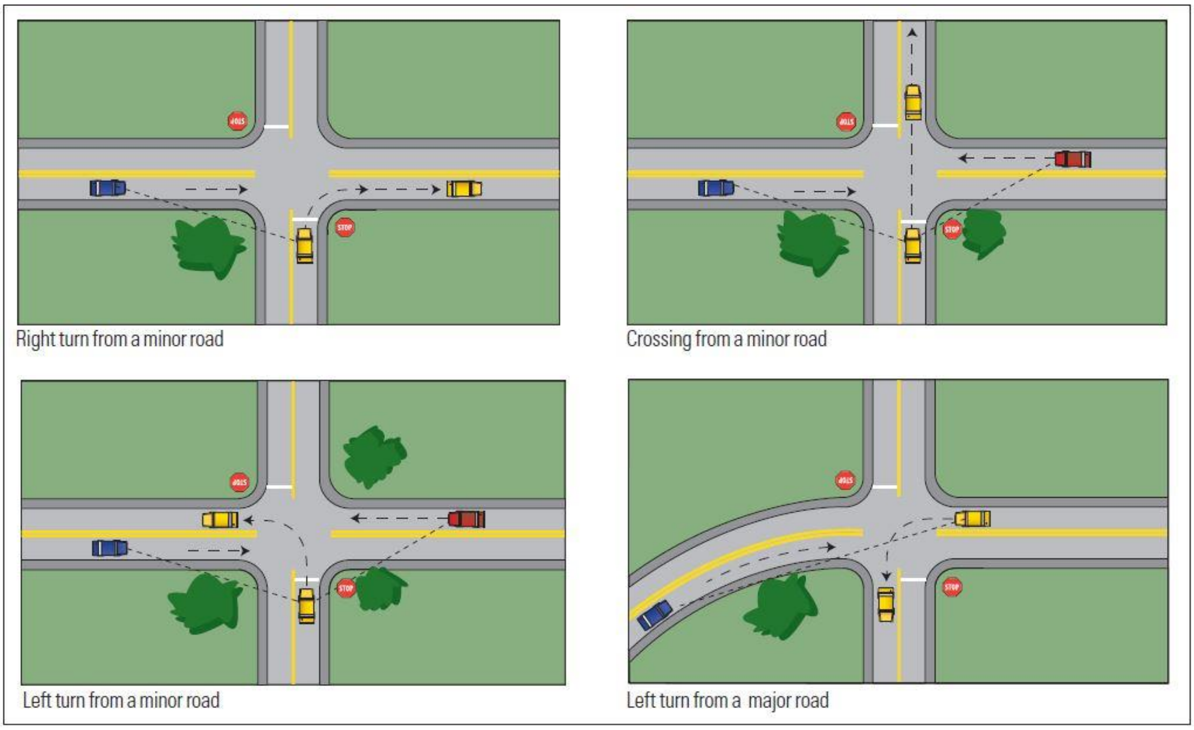


Figure 6.35: Maneuvering Sight Distances at an intersection

→When the volume of the heavy vehicles is high; it may necessary to extend maneuvering gaps to take into account the characteristics of these vehicles (Slower acceleration and deceleration rates, larger sizes). As a result the required sight distances may then be increased significantly, as illustrated in table 6.12 below.

V_{85} MAJOR ROAD (km/h)	REQUIRED SIGHT DISTANCE (m)		
	PASSENGER VEHICLE	TRUCK	SEMI-TRAILER
50	95	150	195
60	115	180	235
70	135	210	275
80	150	240	315
90	170	270	355
100	190	300	395

Table 6.12: Required Sight Distance (Left turn from minor road) (Source: Transportation Association of Canada, 1999)

<u>6.3.2.3 Sight Triangle:</u>

The available sight distance must be sufficient to enable drivers to detect in advance vehicles approaching on adjacent legs with which they could be in conflict. The roadsides of the intersections should be permanently free of any sight obstruction. At a conventional intersection, the area that needs to be free forms a **sight triangle**.

The dimensions of this triangle vary according to:

-The type of intersection (conventional or roundabout)

-The type of traffic control (none or yield)

-The vehicles approach speed

-The assumptions relating to divers behaviors (reaction time, deceleration rate)

●Conventional intersections:

At uncontrolled conventional intersections, the required size of the sight triangle for the vehicle 1 driver is defined by the lengths of D1-D2 and D1 –D3, as shown in figure 6.36.

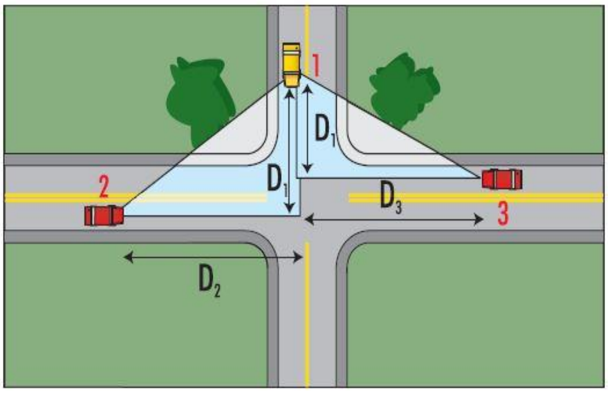
●Roundabouts:

At roundabouts, the dimensions of the pseudo-sight triangle are determined by the D1, D2 and D3 distances as show in figure 6.36:

-D1: Distance on the leg considered; 15 m is generally recommended to avoid excessive approach speeds.

-D2: Required distance between vehicle 1 and a vehicle 2 approaching from the adjacent upstream leg.

-D3: Required distance between vehicle 1 and a vehicle 3 approaching from the ring lane.



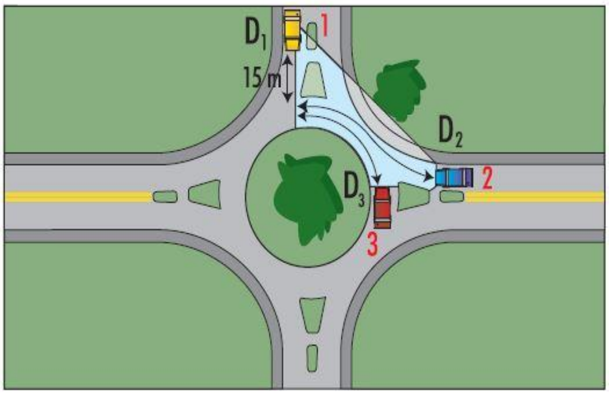


Figure 6.36: Sight Triangles at a conventional intersection and a roundabout

(Right hand side driving)

6.3.2.4 Decision Sight Distance (Intersections and Sections):

Some countries use the decision sight distance criteria in more complex or unexpected driving situations. This criterion provides for an additional safety margin over and above the stopping sight distance (Table 6.13).

	DESIGN SPEED (km/h)				
	70	80	90	100	110
Stopping sight distance (m)	110	140	170	210	250
Decision sight distance (m)	200	230	275	315	335

Table 6.13: Comparison of stopping sight distance and decision sight distance (Source: Transportation Association of Canada, 1999)

6.3.3 Sections:

In sections, most sight distance problems are related to the presence of horizontal or vertical curves.

6.3.3.1 Stopping Sight Distance:

As previously mentioned, the sight distance, at any point of the road network, must be sufficient to a driver travelling at a reasonable speed (V85) to stop his vehicle safely before hitting a stationary object in his path.

6.3.3.2 Passing Sight Distance:

The passing sight distance is the distance that a driver has to see ahead of him in the incoming lane to be able to complete safe passing maneouvre. This distance is required on two-way, two-lane roads (Of course where the pavement marking allows passing).

The maneouvre can be broken down into four stages: Perception and reaction, passing maneouvre, safety margin and distance travelled by the incoming vehicle (Figure 6.37).

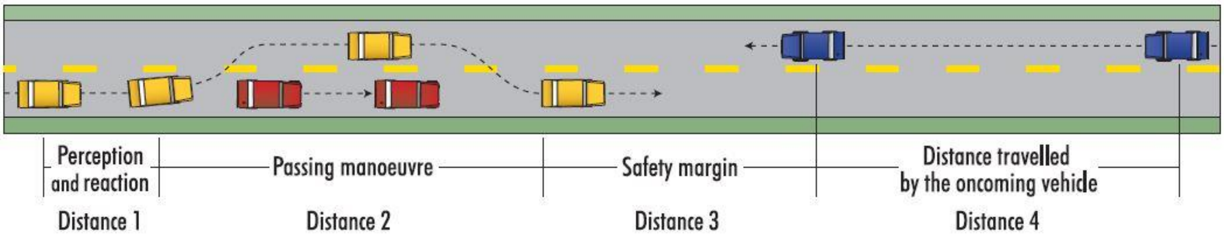


Figure 6.37: Passing Manoeuvre

The required passing sight distance may vary significantly from a country to another depending on the assumptions made at each stage (Table 6.14).

	SPEED (km/h)								
Country	50	60	70	80	90	100	110	120	130
Australia	330	420	520	640	770	920	1100	1300	1500
Austria	-	400	-	525	-	650	-	-	-
Canada	340	420	480	560	620	680	740	800	-
Germany	-	475	500	525	575	625	-	-	-
Greece	-	475	500	525	575	625	-	-	-
South Africa	340	420	490	560	620	680	740	800	-
United Kingdom	290	345	410	-	-	580	-	-	-
USA	345	407	482	541	605	670	728	792	-

Table 6.14: Recommended passing sight distances in some countries (Source: Harwood et al. 1995)

6.3.3.3 Meeting Sight Distance:

Some countries use the meeting sight distance as a criterion. This is the distance required for two vehicles coming towards each other to stop without colliding. This sight distance should be considered when two-way traffic is allowed but the road is too narrow for cars to meet safely (e.g. narrow bridge).

The required meeting sight distance is calculated by adding together the stopping sight distances of both vehicles as shown in the figure 6.38 below.

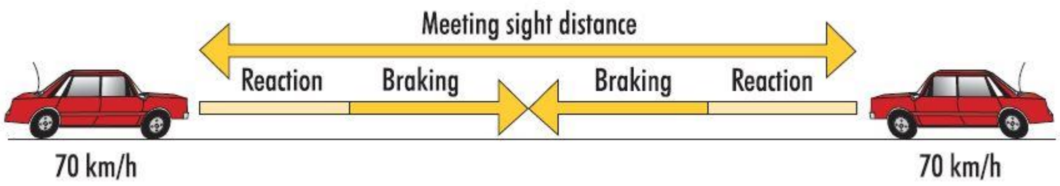


Figure 6.38: Meeting Sight Distance

6.4 Intersections:

6.4.1 Introduction:

Intersections are considered as critical points of a road network as regard safety, capacity and level of service. In this part of a road, drivers can change their path, thus, it enables a large amount of destinations to be reached.

"*Speed at intersections is lower than in its approaches: Sometimes vehicles must even stop*."(Road Safety Manual, PIARC 2003).

Engineers while designing intersections should cater to all drivers' types such as non-expert, unfamiliar and elderly persons. Moreover, design and signing have to provide the correct information at an adequate time and place, for that, signing should be considered from the earliest stages of design.

6.4.2 Choice of intersection type:

The choice of intersection type should be adapted to the relative importance of traffic volumes: Less risk should correspond to larger volumes. Besides, risk should not be excessive even for small traffic volume. In fact, the choice of an intersection design depends on several factors like, the road type and function, the design speed, adjacent land use, traffic volume and type, the cost, the environmental concerns and other network considerations…

6.4.2.1 Choice of intersection according to road's type:

The choice of intersection depends on the road's type. In fact, types not to be used are the following:

*On main rural roads:

- Right-hand priority intersections.

- Signalized intersections.

*On freeways:

-All types of intersections or roundabouts

6.4.2.2 Choice of intersection according to environment:

*Rural main roads:

The big problem related to the choice of intersection type on rural main roads is the problem of drivers' priority. In fact, annually it causes several fatal accidents due to its operation principle.

' On rural main roads, fixed signed priority intersections have a rather poor safety level, precisely because of their operation principle: priority drivers, driving fast and conscious of their priority, interact with non-priority drivers for whom crossing the priority road represents a delicate task. Information acquisition, treatment, decision and maneuvering are subjected to strong time limitations. When traffic volumes are high, accident frequencies are also high and a roundabout may be a solution' (Service d'études techniques des routes et autoroutes/centred'étude des transports urbains, 1992).

*Rural secondary roads:

Generally, with rural secondary roads, the main used intersections are:

-Right hand priority

-Fixed signed priority

-Roundabouts: Especially where traffic volumes are noticeable (Safety problems).

*Rural highways:

Eligible intersection types are:

-Signed priority intersections (STOP or YIELD).

-Roundabouts: Except on divided highways with more than two lanes in each direction, since continuity is broken.

* Bypass roads:

When bypassing villages, transverse traffic can be high. Accidents concentrate especially at intermediate intersections (more than 70% of personal injury accidents occur there). Thus, some precautions should be taken:

-Minor intersection should be suppressed, and if traffic is important, the bypass should be crossed over or under, without connection.

-Roundabout is the most advisable intersection type.

* Urban roads:

Eligible intersection types are:

-Arterials: roundabouts (at major intersections), signalized intersections.

-Collectors: roundabouts, signalized intersections, fixed signed priority intersections and right hand priority intersections.

-Residential streets: roundabouts, right-hand priority intersections.

6.4.2.3 Choice of intersection according to cost:

The term cost is a key point in any construction project. Thus, engineers have to choose the cheapest solution, but of course, which respects the safety norms. For this, roundabouts are considered the best intersection type. In fact, it has an advantage over other types of intersections that land occupation and its construction cost are relatively low.

Also, maintenance cost is lower for roundabouts than for signalized intersections.

6.4.2.4 Choice of intersection according to road's capacity:

The intersection type is strongly depending on the road's capacity. The biggest is the number of vehicles passing from the road per time's unit; the biggest is the intersection size (and type). Moreover, signalized intersections are the intersection type that assumes the biggest number of vehicles. Thus, it is the best solution for big traffic zones.

The table 6.15 below indicates the approximate capacity of several types of intersections:

INTERSECTION TYPE	CAPACITY (pcph)
Right-hand priority	1,000 – 1,500
Fixed-priority	5,000 –12,000
Single-lane roundabout	20,000 – 28,000
Multi-lane roundabout	35,000 – ?[a]
Signalized intersection	20,000 – 80,000[b]

Table 6.15: Intersection type based on capacity (a: depending on countries / b: depending on the lane assignment)

The figure 6.39 below indicates the range of application of different types of intersections (Exp: England):

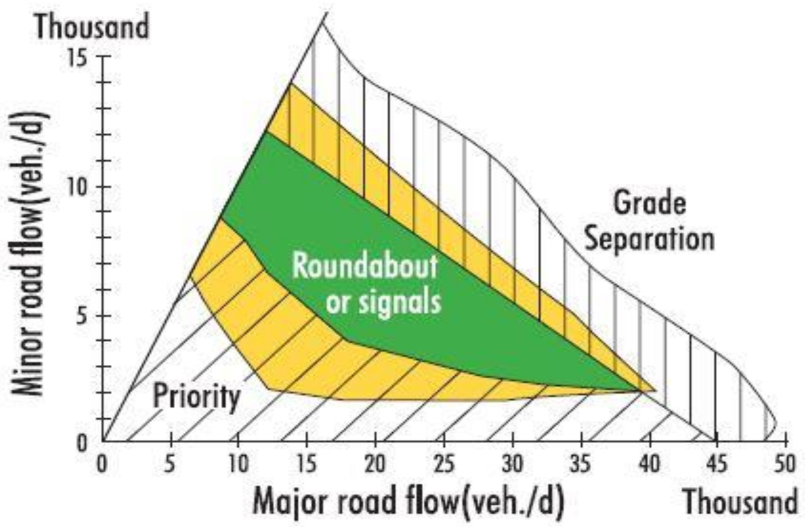


Figure 6.39: Intersection type based on traffic low

6.4.3 Design principles for intersections:

In intersections design there are some differences between urban and rural areas:

6.4.3.1 Residential intersections:

Generally, accessibility is more important than mobility and capacity in residential streets. The convenience for residents is that there is less pollution, lower mobility and a greater safety for pedestrians. To operate safely residential streets should follow the following principles:

-The street network in the residential area should be either discontinuous or circuitous.

-The number of direct connections to arterial or main streets in limited and achieved mainly through collector streets.

→If we want to improve residential street networks to reduce the traffic volume, we have to be careful. Otherwise, we have to ensure that, by solving existing problems, we are not only moving them to the next residential area.

6.4.3.2 Urban intersections:

At urban intersections the following points should be taken into account:

-Driveway problems next to the intersection should be solved.

-Pedestrian activity should be foreseen.

-Needs of all types of road users should be properly addressed (Cyclists, transit...).

-Additional lanes should operate independently from through lanes.

6.4.3.3 Rural intersections:

At rural intersections the following points should be taken into account:

-An adequate sight distance should be provided at each point of the road's network and at the intersection itself.

-Strict approach designs should be avoided: tight curves, high grades.

-Design should be based on operational speed in the priority highway.

If right hand priority intersections are allowed on the secondary road network, the following measures should be adopted:

-Increase of sight distance: parking prohibition, offsetting fences and greenery.

-Reinforcement of the intersection presence: clearing of the roadsides, colored pavements, destination signings…etc.

6.4.3.4 Roundabouts:

Roundabouts are mainly designed for high traffic volumes. However, in urban or suburban areas, traffic conditions may not always be compatible with roundabout operations.

6.4.4 Distance between intersections:

Generally, the distance between adjoining intersections has an effect on the road's safety and, therefore, its service level. For instance, in highways the distance between intersections should be a great distance to assure mobility. Moreover, a short distance between intersections leads to increase the accident rate.

However, in urban and suburban areas, keeping an ideal distance between intersections becomes a hard mission especially if land use is high. In arterial streets, the distance between intersections should be uniform and above 200[6] m, also, it is possible to improve the situation by prohibiting some left turns and making one-way arterials.

On local streets, the minimum distance between adjoining intersections should be 60 m for 4-leg intersections and 40 m for 3-leg intersections. It is 60 m on a collector street.

6.4.5 Conflict points at intersections:

One of the most important tasks for an intersection is to assure for any vehicle to cross and change direction safely. Thus, it has a set of conflict points between vehicle paths that may lead to an increase of accident rate, but designers should, always, think to minimize the severity of potential accidents at these points and the interactions related to it which can be classified as **convergence**, **divergence** and **crossing**.

To make safer intersections, road designers should minimize as possible the number of conflicts. For instance, 3 leg-intersections are safer than 4-legs ones and more than 4-legs must be substituted by roundabouts. Also, the number of conflict points may be reduced by suppressing non-priority movements.

[6] Distance to right-only-intersections can be lowered to 100 m (Right-hand driving countries).

The general rule in intersections is that the number of conflict points is well related to the intersection number of legs. In fact, it increases rapidly with the number of intersection legs. Figure 6.40 below, illustrates the foregoing.

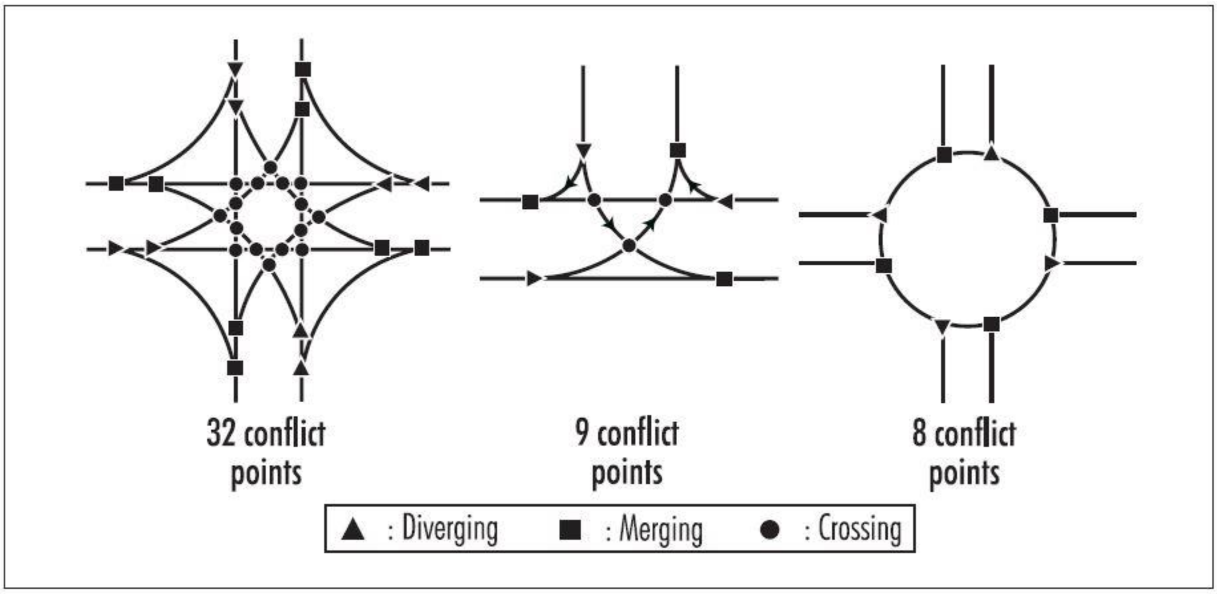


Figure 6.40: Number of conflict points at intersections and roundabouts

The distance between conflict points should be a great distance by means of traffic islands and/or auxiliary lanes. Also, signalized intersections can be used: There is a time separation which reduces the need for space separation.

Angles at which conflicts occur should be properly managed:

-For crossing manoeuvres, the angle between paths is between 75° and 105° (It improves visibility and reduces crossing distances).

-For convergence and divergence manoeuvres, angle is less than 5°.

-For insertion manoeuvres, the angle is between 20° and 60° (To control the insertion speed).

6.4.6 Special road users:

6.4.6.1 Pedestrians:

In urban and suburban areas, pedestrian paths should always be as short as possible. However, provision of pedestrian crossings (delineated by zebra stripes) run-over accidents because it shows pedestrians the safest path to cross and warns the drivers of the possibility of conflict with pedestrians.

When the crossed width is considerable (around 10 m), the efficiency of pedestrian crossings becomes lower (by more than 60%). Thus, in this case a central refuge island can be a solution so that pedestrians can cross in two phases. Also, narrowing the roadway at the pedestrian crossing by enlarging sidewalks or making a contrasting pavement (cobbled or made of a different colors). Moreover, the pedestrian crossings should be near to intersections and to make them noticeable and enhance their visibility by increasing their numbers.

In roundabouts, crossing of the central island should be always avoided. Besides, segregated pedestrian paths are desirable to reduce the crossed distance.

Disabled pedestrians require special measures like a gap in the curb for wheelchairs with neither slopes higher than 10 % nor steps above 10 mm. Also, a textured pavement recognized by blind people and frontier to the roadway should marked by a step of 10 mm.

6.4.6.2 Heavy vehicles:

Heavy vehicles are considered special road users especially when it is in small intersection radii which may increase encroachment and, therefore, means increasing in number of fatal accidents. Usually, the main causes for these accidents are the sharp changes in superelevation; sharp turns at roundabout exits; also too small path deflection at the entrance (with high speeds) and finally the long straight sections in the roundabout lane, terminating in short-radius curves.

Figure 6.41: Heavy vehicle encroaching on opposing lane when turning at intersection

6.4.6.3 Transit:

Usually, bus stops tend to be located near intersections so that their customers have an easier access to a larger number of their destinations. Stops located after the intersections facilitate bus re-entry into normal traffic. In roundabouts, they can be located off-road, both before the entrance and after the exit.

6.4.6.4 Two wheelers at roundabouts:

At roundabouts, the cyclists account nearly 50 % of injury accidents (too much higher rate than cars). Two wheelers drivers while increasing their radius paths may hamper their vision field by their helmet. However, where a large number of cyclists is expected, a different type of intersection should be considered, split-level routes both for pedestrians and cyclists, and also alternative routes off the roundabout should be adopted in order to reduce accident rate.

6.4.7 Road Alignment:

6.4.7.1 Vertical Alignment:

Generally, an ideal approach for an intersection should never have grades more than 6 % and not over 3 % in order to improve the visibility and the comfort of vehicle having to stop at the intersection and also to enable drivers to correctly evaluate needed speed changes. Moreover, intersections should not be located near crest vertical curves.

Figure 6.42: Hazardous combination: Hill, intersection, accesses, horizontal curve

Usually, to an intersection and inside it there should not be large variations in grade. For instance, for speeds higher than 70 km/h grade difference between ends of a vertical curve should not exceed 2 %. Besides, for a 50 km/h speed, the difference can reach 4 % if the visibility is sufficient (Comfort is loss but not safety). However, for a 30 km/h speed, the difference can be as high as 6 %.

‘*Vertical curves should not reach to less than 20 m from the common pavement zone, this distance can be reduced (to 10 m and even to 5 m) if the intersection carries little traffic’* (Road Safety Manual PIARC 2003).

6.4.7.2 Horizontal Alignment:

The ideal location for an intersection is on a tangent because location in curves may cause problems. In fact the visibility is reduced; also the superelevation and lane widening make the situation more complicated. Besides, there will less friction for a vehicle to brake since a fraction of the available skid resistance is already consumed to keep the vehicle on a curved path. Moreover, there is a larger conflict potential for vehicles trying to cross a priority road.

Roundabouts should never be placed on a curve as it may create visibility and orienteering problems. Indeed, it is possible in sometimes to replace a curve by a roundabout; both tangents are maintained and the change in direction takes place inside the roundabout.

6.4.8 Safety at intersections:

6.4.8.1 Safety at roundabouts:

Roundabouts have to enable vehicles to change legs safely. Despite the low number of accidents in roundabouts comparing to other types of intersections, this advantage may be hindered by excessive traffic volume or speed (Fatalities at roundabouts are fewer than other types of intersections since accidents are less severe).

However, special attention should be paid for their design. For instance, the pedestrians and cyclists can pose problems. Also, too-large central islands (more than 30 m in diameter) are less safe, and to pay a special attention to the curvature of the path of entering vehicles.

Actually, two cases can be discussed concerning safety at roundabouts: **rural and suburban areas**, and also **urban areas**. In fact, concerning roundabouts at rural and suburban areas, the main accident type is loss of control at the entrance of the roundabout so the running to the central island (nearly 40 % of personal injuries). This loss may be caused by the surprise of some travelers who knew the intersection before making it a roundabout. Also, with an aggressive design, there is usually a sharp deceleration on the central island which causes fatal accidents. Moreover, other types of accidents may occur in such areas, like losses of control in the ring lane especially if it is elliptical and the collision between entering vehicles and those traveling on the ring lane.

However, in urban areas accidents occur especially between two wheelers and heavy vehicles when an entering vehicle meets another travelling on the ring lane. Besides, other types of accidents may occur like the loss of the control at the entrance or in the ring lane (especially for motorcyclists), and pedestrians run-overs when they try to cross an entrance or an exit or also when pedestrians want to cross the ring lane to cut across a too large roundabout.

Figure 6.43: Roundabouts at Urban and Rural Areas

6.4.8.2 Safety at Signalized intersections:

*Left-turn accidents:

The main safety problems related to left-turn maneuvers are the high speed, the storage difficulties, the poor perception of opposing vehicles especially two wheelers, the poor estimation of remaining time to coincide with the opposing vehicle and also the difficulties related to left-turners to find a correct transverse placement.

‘Lengthening the ‘all-red’ phase increases the risk of left-turn accidents, since the number of vehicles turning on red increases, too. ‘All-red’ phase (plus amber) should allow the intersection to be cleared before the onset of green: Shortening the ‘all-red’ phase should entail lengthening the amber one. This is dangerous because some vehicles will accelerate on amber when they should instead be braking’ (Road Safety Manual PIARC 2003).

*Right-angle Collision:

Right-angle collisions are very severe. It may be caused by red light running or inadequate signal timing. Besides, it may occur due to excessive speed, poor compliance to signal-controlled operation (Moped drivers), belief that there is still time to pass amber and the poor perception of the presence of signals (Which may be caused by speed, road environment or glare).

Moreover, intersection design and signal phasing may contribute to this type of accidents by making excessive width of intersection or short cycles. In fact, if the cycle length is reduced from 120s to 30s, the overall accident frequency is multiplied by 2 and the right-hand accident frequency is multiplied by 4. However, signal and signal phasing may also contribute to reduce the accidents; In fact, some approaches leads to reduce accident frequency like making central refuge island, reduction of the ‘all-red’ phase to minimum and also with some layouts to encourage moderate speed such as a slight deflection of through paths by an offset of the opposing accesses.

*Pedestrian Run-overs:

Pedestrian accidents frequency at signalized intersections is relatively high. Usually it happens when a pedestrian wants to cross a lane and a vehicle passes. Most of cases happen when pedestrian has a red signal.

Run-overs of elderly persons are to be especially catered because the fatality of accidents is much higher with old persons than young ones. Run-overs take place usually when starting to cross, with vehicles leaving the intersection, turning left or turning right on amber also, with backing up vehicles. Moreover, between the factors that increase run-over risk there is the poor maintenance of signals, when pedestrian volume is high and speeds are low there is a possibility of vehicles turning right on a flashing amber, also the large distance between the pedestrian crossing and the adjoining intersection, and the large number of lanes crossing the intersection. Indeed, four lanes instead of one multiplies the risk by 2, 5.

6.4.8.3 Safety at transformation of Right-hand to fixed-signed priority intersections:

Transformation of right-hand priority intersections into fixed-signed priority ones causes always an increase of accident frequencies especially with some environmental conditions like roadway width (narrow) and the traffic volume in this zone.

‘Experiences in the transformation of right-hand priority intersections into fixed-signed priority ones show some increase in accidents with high traffic volumes, especially when the roadway is narrow or at the traversing of small villages’ (Service d’études techniques des routes et autoroutes/Centre d’études des transport urbains, 1992).

6.4.8.4 Safety at transformation of priority to signalized intersections:

Transforming three-leg fixed-signed priority intersections into signalized ones does not improve safety significantly. However, transforming four-leg fixed-signed priority intersections into signalized ones reduces significantly both the number of crashes and their severity. Besides, transforming four-leg right-hand priority intersections into signalized ones reduces significantly the number of accidents, but not their severity.

This difference in properties is essentially due to the difference in shape between three and four-leg intersections, and also to the differences in speed between right-hand priority and signs.

6.4.8.5 Safety at Four-leg + intersection:

For safety reasons, four-leg intersections should only be allowed on low volume highways or where most traffic approaching from the non-priority road turns, instead of crossing the priority road. Actually, four-leg + intersections should be avoided in rural divided roads, since non-priority vehicles have to cross a large width. The accident risk of such crossings is 1, 5 times for the crossing of an undivided road, 10 times for traveling 1 km on an undivided road and can reach 30 times for crossing a roundabout.

7 Alignment Design of a New Rural Road in the Province of Bologna:

7.1 Project Location:

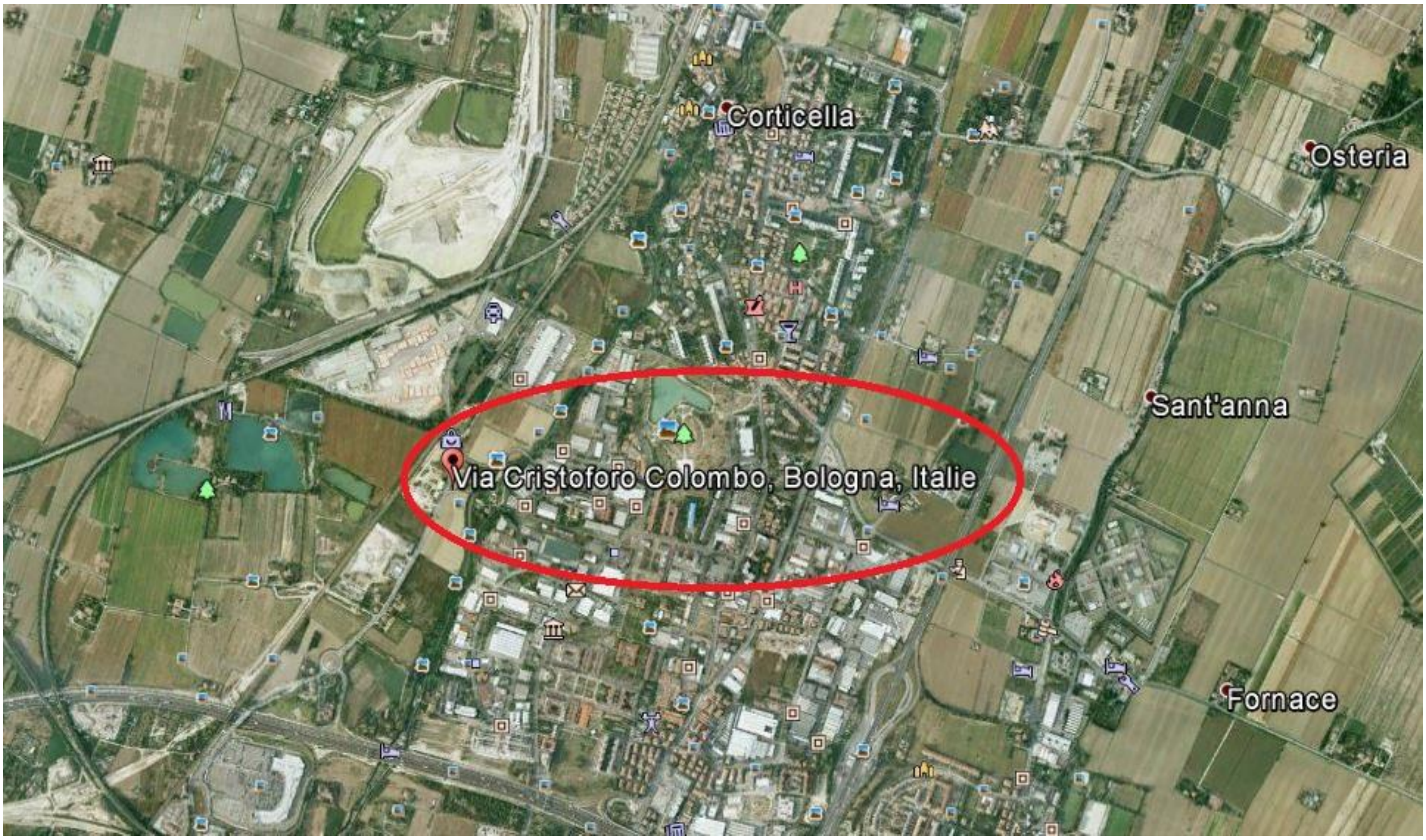


Figure 7.1: Project Location

This road is located in the province of Bologna-Italy (‘Comune di Argelato’, 26 km in the north of Bologna), and has 5677, 81 m of length. It connects the intersection ‘Via Rosario-Via Corticella’ (Via Cristoforo Colombo-Comune di Bologna) to the ‘Strada Provinciale SP3’ (Comune di Argelato) passing from ‘Via Ronco’, ‘Via Lirone’ and ‘Via Bondanello’.For this, it has three main intersections.

The overview in the figure 7.2, gives the general form of the road from the starting point to the arriving one including the main intersections.

Figure 7.2: Project Overview

The following three plans show the three main intersections of the road where was necessary to make bridges. Indeed, the plan1 shows the intersection with 'Via Ronco'. The second one is the intersection with 'Via Lirone' and the last one shows the intersection with 'Via Bondanello'. Moreover, we have some other intersections ('Via pioppe' and 'via Muraglia') where underpasses were made (Sottopasso agricolo).

Figure 7.3: Project plan 1: Intersection at Via Ronco

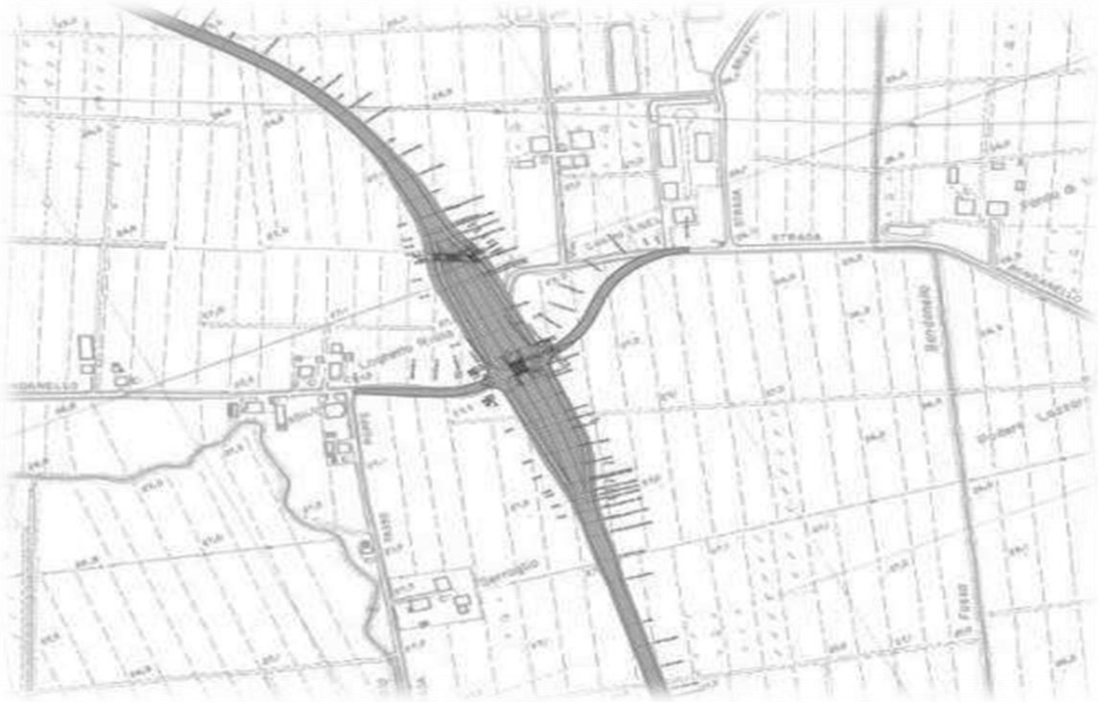


Figure 7.4: Project Plan 2: Intersection at Via Lirone

Figure 7.5: Project Plan 3: Intersection at Via Bondanello

•Problem at the road entrance :

As shown in the Figure 7.6 below, the entrance roundabout of the new designed road seems having a strange form. In fact, it is not a real roundabout: The green zone is not yet used. This is due to property problems. Otherwise, the province of Bologna is not the owner of this zone, thus it couldn't use it to make a suitable road entrance.

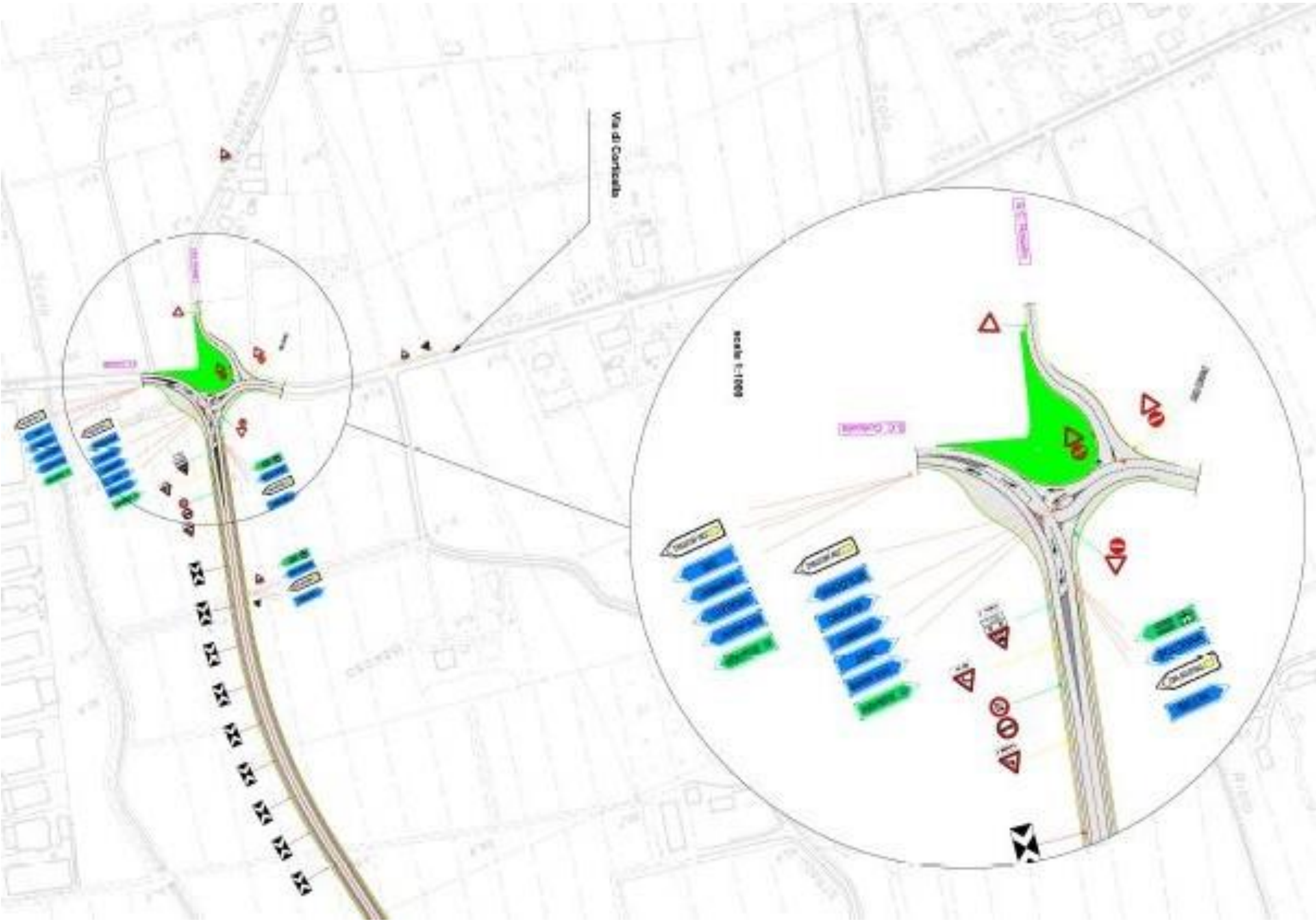


Figure 7.6: Road Entrance

7.2 Vertical and Horizontal Alignment:

Horizontal and vertical alignments establish the general character of a rural highway, perhaps more than any other design consideration. The configuration of line and grade affects safe operating speeds, sight distances, and opportunities for passing and highway capacity. Decisions on alignment have a significant impact on construction costs.

7.2.1 Vertical Alignment:

Generally, the alignment of a road is a three-dimensional problem measured in x, y and z dimensions. The horizontal alignment is shown in x and z coordinates (Called Plan View), and the Vertical alignment is shown in the Y axis (Called Profile View). This latter, specifies the elevations of points along a road. Elevations are determined by need to provide proper drainage and driver safety. In addition, a primary concern for a vertical alignment is to establish a transition between two roadway grades by means of a vertical curve (Crest or Sag).

Some vertical curves include:

●The initial road grade is called G1 the final road grade is called G2 and is typically given in percent.

●PVC is the point of the vertical curve.

●The point of intersection of the initial tangent grade and the final tangent grade is the point of vertical intersection (PVI).

●The absolute value of the difference between G1 and G2 is called A, and is given in percent.

●The point of intersection of the vertical curve with the final tangent grade is called the PVT.

●The length (L) of the vertical curve is the horizontal distance between PVC and PVT.

7.2.2 Horizontal Alignment:

Some of the factors that influence the location and configuration of the horizontal alignment include:

●Physical controls: topography, watercourses, geophysical conditions, land use, and man-made features.

●Environmental considerations: affect on adjacent land use, community impacts, ecologically sensitive areas.

●Economics: construction costs, right-of-way costs, utility impacts, operating and maintenance costs.

●Safety: sight distance, consistency of alignment, human factor considerations.

●Road classification and design policies: functional classification, level of service, design speed, design standards.

⟶Although the designer must attempt to optimize the horizontal alignment with respect to these factors, the alignment **cannot be** finalized until it has been compared and coordinated with the vertical and cross-sectional features of the road.

7.2.3 Horizontal and Vertical Alignment Coordination:

The horizontal alignment cannot be finalized until it has been compared and coordinated with the vertical and cross-sectional features of the road. Horizontal curvature and vertical grades should be in proper balance. Emphasis on the horizontal tangent alignment is not desirable when it results in extremely steep or long vertical grades. Neither is emphasis on flat vertical grades when it results in excessive horizontal curvature. A compromise between the two extremes is the best approach. Several general criteria should be kept in mind:

-Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition makes it difficult for drivers to perceive the horizontal change in alignment, especially at night.

-Sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve. This is aesthetically undesirable and can be hazardous since vehicle operation speeds, particularly trucks, are often higher at the bottom of vertical grades.

-On two-lane roads and streets with considerable traffic volume, safe passing sections must be provided at frequent intervals and for an appreciable percentage of the length of the roadway. In these cases it is necessary to work toward long horizontal tangent sections to secure sufficient passing sight distance rather than the more economical combination of vertical and horizontal alignment.

-Both horizontal curvature and the vertical profile should be as flat as feasible at intersections where sight distances along both roads and streets is important and vehicles may have to slow or stop.

The horizontal and vertical alignments should be **in balance** as shown in figure 7.7 below. A generous flowing alignment in one plane is not compatible with small and frequent breaks in the other.

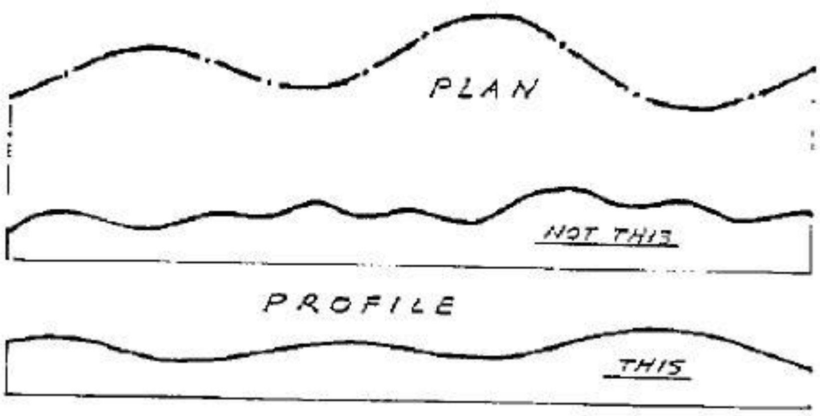


Figure 7.7: Balanced Vertical and Horizontal Alignment

7.3 Alignment Design of the Rural Road:

See the AutoCAD Files in the attached Annex A

7.4 Safety Verification:

7.4.1 Vertical Alignment:

	C.Type	G1	G2	R (m)	VPC	VPI	VPT	L (m)		K	SSD (m)	V (km/h)
1	Sag	-0,05%	2,92%	4000	400,59	460	519,38	118,79		40	160	90
2	Crest	2,92%	0,57%	4000	533,16	580	626,85	93,69		40	160	90
3	Crest	0,57%	-3,06%	4000	917,35	990	1062,62	145,27		40	160	90
4	Sag	-3,06%	-0,07%	4000	1180,18	1240	1299,85	119,67		40	160	90
5	Sag	-0,07%	2,66%	3600	1415,9	1465	1514,08	98,18		36	130	80
6	Crest	2,66%	-2,88%	6000	1518,6	1684,79	1850,98	332,38		60	185	100
7	Sag	-2,88%	-0,04%	3600	1855,95	1907	1958,07	102,12		36	130	80
8	Sag	-0,04%	3,56%	4000	2428,03	2500	2571,92	143,89		40	160	90
9	Crest	3,56%	-2,20%	4000	2587,25	2702,4	2817,59	230,34		40	160	90
10	Sag	-2,20%	1,04%	4000	2882,5	2947,4	3012,31	129,81		40	160	90
11	Crest	1,04%	-3,03%	4000	3163,57	3245	3326,39	162,82		40	160	90
12	Sag	-3,03%	-0,06%	4000	3350,68	3410	3469,35	118,67		40	160	90
13	Sag	-0,06%	3,01%	3700	3594,81	3651,7	3708,57	113,76		37	130	80
14	Crest	3,01%	-1,03%	3900	3769,29	3848,13	3927	157,71		39	160	90
15	Crest	-1,03%	-2,97%	4000	3971,19	4010	4048,79	77,6		40	160	90
16	Sag	-2,97%	-0,09%	4000	4137,42	4195	4252,61	115,19		40	160	90
17	Crest	-0,09%	-0,97%	6000	4603,54	4630	4656,45	52,91		60	185	100
18	Sag	-0,97%	-0,02%	6000	4842,71	4871,31	4899,91	57,2		60	220	110
19	Crest	-0,02%	-3,79%	2475	5119,51	5166,15	5212,8	93,29		24,75	105	70
20	Sag	-3,79%	4,44%	2107	5212,9	5299,65	5386,4	173,5		21,07	85	60
21	Crest	4,44%	-0,04%	2150	5412,35	5460,6	5508,85	96,5		21,5	105	70

Table 7.1: Vertical Alignment Verification

The table 7.1 above shows the safety verification of the vertical alignment. The green column separates the collected data from the calculated one. Infact;

- G1 & G2= tangent grades in percent.

- R: radius of vertical curve.

- VPC= vertical point of curvature.

- VPI= vertical point of intersection.

- VPT= vertical point of tangency.

- L= Length of the vertical curve.

- K= rate of vertical curvature.

- SSD= stopping sight distance.

- V= speed.

Where: **A= |G1-G2|** (Equation 12)

K=L/A (Equation 13)

→From the value of **K**, we can get the Stopping Sight Distance (**SSD**) and the Speed (**V**) from AASHTO as follow:

Metric			
Design speed (km/h)	Stopping sight distance (m)	Rate of vertical curvature, K^a	
		Calculated	Design
20	20	2.1	3
30	35	5.1	6
40	50	8.5	9
50	65	12.2	13
60	85	17.3	18
70	105	22.6	23
80	130	29.4	30
90	160	37.6	38
100	185	44.6	45
110	220	54.4	55
120	250	62.8	63
130	285	72.7	73

Table 7.2: Design Controls for Sag Vertical Curves

Metric			
Design speed (km/h)	Stopping sight distance (m)	Rate of vertical curvature, K^a	
		Calculated	Design
20	20	0.6	1
30	35	1.9	2
40	50	3.8	4
50	65	6.4	7
60	85	11.0	11
70	105	16.8	17
80	130	25.7	26
90	160	38.9	39
100	185	52.0	52
110	220	73.6	74
120	250	95.0	95
130	285	123.4	124

Table 7.3: Design Controls for Crest Vertical Curves

In the curve number 18 (Yellow Row), the calculated speed is 110 km/h which is greater than the design speed (100 km/h). So, it is suggested to make it 100 km/h.

7.4.2 Horizontal Alignment:

	R (m)	e	f	V (km/h)	Sight Distance	SSD	
1	450	6,00%	0,11	100	173	160	OK
2	480	5,00%	0,11	100	175	160	OK
3	450	5,00%	0,11	100	169	160	OK
4	1000	4,02%	0,11	100	200	160	OK
5	800	4,62%	0,11	100	370	160	OK
6	600	5,00%	0,11	100	350	160	OK
7	450	5,50%	0,11	100	240	160	OK
8	490	5,00%	0,11	100	170	160	OK
9	500	5,20%	0,11	100	390	160	OK
10	438	5,50%	0,11	100	212	160	OK
11	400	0,055	0,11	90	180	140	OK
12	740	4,50%	0,11	100	175	160	OK
13	590	4,50%	0,11	100	170	160	OK
14	53,84	5,00%	0,11	60	70	65	OK
15	53,84	5,00%	0,11	60	85	65	OK

Table 7.4: Horizontal Alignment Verification

In the Table 7.4 above, the values of '**e**' and '**R**' are given from the alignment design. However, the friction value '**f**' is taken from the Italian norms as shown in the table 7.5 below:

Velocità km/h	25	40	60	80	100	120	140
aderenza trasv. max imp. $f_{t\ max}$ per strade tipo A, B C, F extra urbane, e relative strade di servizio	-	0,21	0,17	0,13	0,11	0,10	0,09
aderenza trasv. max imp. $f_{t\ max}$ per strade tipo D, E, F urbane, e relative strade di servizio	0,22	0,21	0,20	0,16	-	-	-

Table 7.5: Friction Value (Source: Decreto Ministeriale 5 Novembre 2011)

For a design speed of 100 km/h and for a rural road (Class C: 'Extra Urbane'), we have a friction value f= 0.11

From the Italian norms: If **R** is less than 118 m, then **V** = 60 km/h.

If **R** is greater than 437 m, then **V** = 100 km/h.

But if R is between these two values (**118 m < R < 437 m**), then R is calculated from the formula:

$$R = V^2 / 127(e + f) \qquad \text{(Equation 14)}$$

The only case where it was necessary to use this formula is the case of the curve number 11 (The yellow row in the table 7.4). The result was V = 90 km/h: We passed from V = 100 km/h to V = 90 km/h, it is a range of 10 km/h so it is acceptable.

After getting the Speed value, it is possible to calculate the Stopping Sight Distance using the Italian norms as shown in the figure 7.8 below:

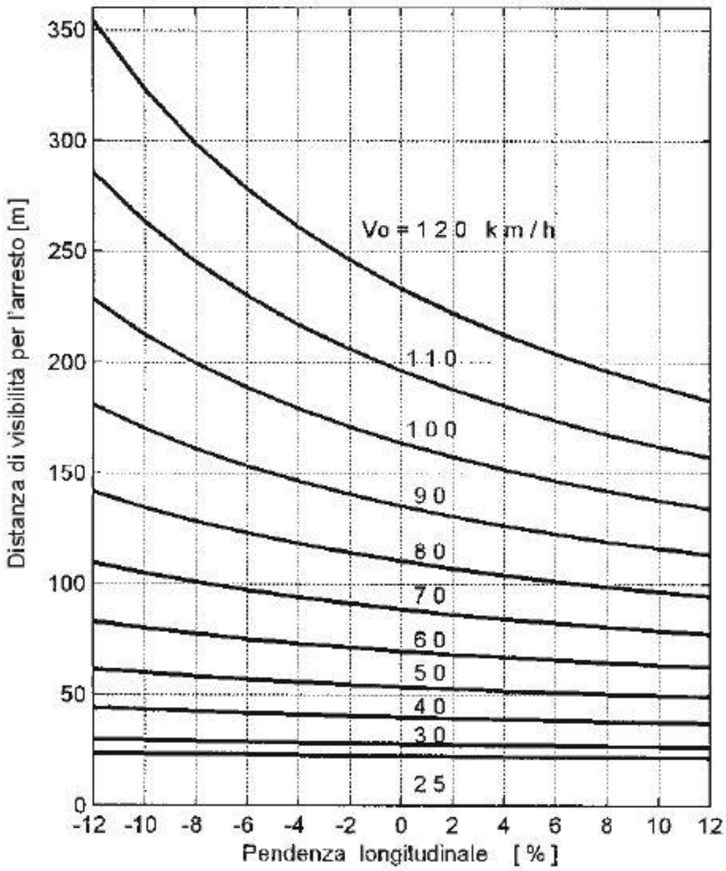


Figure 7.8: The Stopping Sight Distance (Source: Decreto Ministeriale 5 Novembre 2011)

The Sight Distance is calculated graphically, then it is compared to the stopping Sight Distance: If SD > SSD so it is OK. If not; either the curve radius or the design speed should be changed. In our case, SD is always **greater** than SSD so the verification was made and the road is safe.

8 Pavement Design of a New Rural Road in the Province of Bologna using BISAR 3.0:

8.1 Main Principles of the BISAR Program:

In the early 1970s, Shell Research developed the BISAR mainframe computer program, which was used in drawing the design charts of the Shell Pavement Design Manual issued in 1978. An abbreviated version of the BISAR program for use on a personal computer2 was issued in 1987 as BISAR-PC (Release R 1.0). A PC version comprising all extensive mainframe options was not feasible, because of the lengthy calculations at that time, The PC version was issued to facilitate the use of the design charts and to avoid laborious interpolations. To avoid these limitations, the DOS program BISAR-PC 2.0, issued in 1995, offered all the possibilities of the former mainframe program.

With the release of **BISAR 3.0** the full possibilities of the original mainframe BISAR computer program are now available for use in the Windows environment. In addition to the calculation of stresses and strains BISAR 3.0 is capable of calculating deflections and is able to deal with horizontal forces and slip between the pavement layers. This offers the opportunity to calculate comprehensive stress and strain profiles throughout the structure for a variety of loading patterns, including air-crafts.

With the BISAR program, stresses, strains and displacements can be calculated in elastic multi-layer system which is defined by the following configuration and material behavior:

1. The system consists of horizontal layers of uniform thickness resting on a semi-infinite base or half space.
2. The layers extend infinitely in horizontal directions.
3. The material of each layer is homogeneous and isotropic.
4. The materials are elastic and have a linear stress-strain relationship.

The system is loaded on top of the structure by one or more circular loads, with a uniform stress distribution over the loaded area.

BISAR calculations require the following input:

- The number of layers

- The Young's moduli of the layers

- The Poison's ratios of the layers

- The thickness of the layers (except for the semi-infinite base layer)

- The interface shear spring compliance at each interface

- The number of loads

- The co-ordinates of the position of the center of the loads

- One of the following combinations to indicate the vertical normal component of the load

- Stress and load

- Load and radius

- Stress and radius

- The co-ordinates of the positions for which output is required.

8.2 Pavement Used in the Design: Flexible Pavement:

Flexible pavements are so named because the total pavement structure deflects, or flexes, under loading. A flexible pavement structure is typically composed of several layers of material. Each layer receives the loads from the above layer, spreads them out, and then passes on these loads to the next layer below. Thus, the further down in the pavement structure a particular layer is, the fewer loads it must carry. Figure 8.1 below:

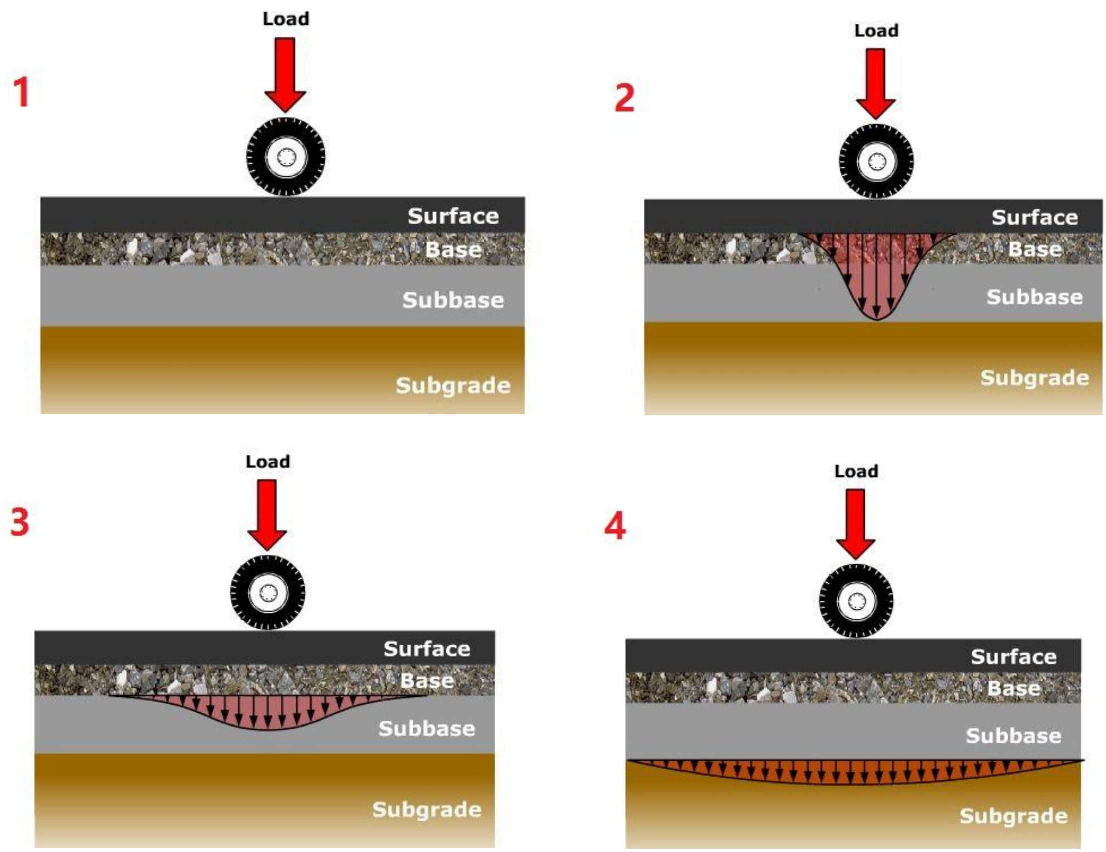


Figure 8.1: Flexible Pavement Load Distribution

In order to take maximum advantage of this property, material layers are usually arranged in order of descending load bearing capacity with the highest load bearing capacity material (and most expensive) on the top and the lowest load bearing capacity material (and least expensive) on the bottom. The typical flexible pavement structure consists of:

- *Surface course*: This is the top layer and the layer that comes in contact with traffic. It may be composed of one or several different HMA sub-layers.
- *Base course:* This is the layer directly below the HMA layer and generally consists of aggregate (either stabilized or unstabilized).
- *Sub-base course:* This is the layer (or layers) under the base layer. A sub-base is not always needed.

Each of these layers contributes to *structural* support and drainage. The surface course (typically an HMA layer) is the stiffest and contributes the most to pavement strength. The underlying layers are less stiff but are still important to pavement strength as well as drainage and frost protection. A typical structural design results in a series of layers that gradually decrease in material quality with depth.

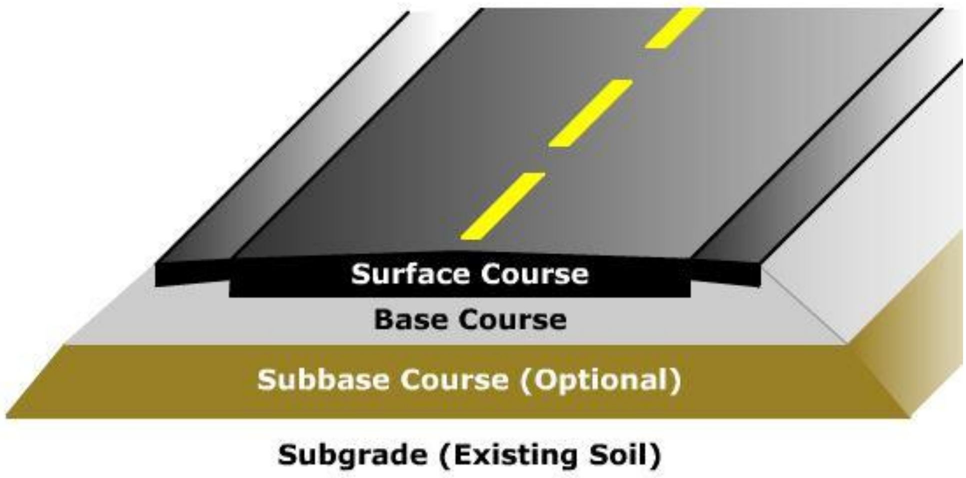


Figure 8.2: Basic Flexible Pavement Structure

The surface course provides characteristics such as friction, smoothness, noise control, rut and shoving resistance and drainage. **The base course** is immediately beneath the surface course. It provides additional load distribution and contributes to drainage and frost resistance. **The sub-base** course is between the base course and the subgrade. It functions primarily as structural support but it can also improve drainage, minimize the intrusion of fines from the subgrade into the pavement structure, and provide a working platform for construction.

8.3 Results and Interpretations:

8.3.1 Results:

BISAR 3.0 - Block Report

MAHER Ayat Thesis

System 1: (untitled)

Structure

Layer Number	Thickness (m)	Modulus of Elasticity (MPa)	Poisson's Ratio
1	0,030	3,000E+03	0,35
2	0,040	2,500E+03	0,35
3	0,300	5,000E+02	0,35
4		5,000E+01	0,35

Loads

Load Number	Vertical Load (kN)	Vertical Stress (MPa)	Horizontal (Shear) Load (kN)	Horizontal (Shear) Stress (MPa)	Radius (m)	X-Coord (m)	Y-Coord (m)	Shear Angle (Degrees)
1	3,000E+01	7,479E-01	0,000E+00	0,000E+00	1,130E-01	-1,500E-01	0,000E+00	0,000E+00
2	3,000E+01	7,479E-01	0,000E+00	0,000E+00	1,130E-01	1,500E-01	0,000E+00	0,000E+00

Position Number	Layer Number	X-Coord (m)	Y-Coord (m)	Depth (m)	Stresses XX (MPa)	Stresses YY (MPa)	Stresses ZZ (MPa)	Strains XX µstrain	Strains YY µstrain	Strains ZZ µstrain	Displacements UX (µm)	Displacements UY (µm)	Displacements UZ (µm)
1	1	1,500E-01	0,000E+00	0,000E+00	-1,779E+00	-1,992E+00	-7,479E-01	-2,733E+02	-3,691E+02	1,906E+02	-2,278E+01	0,000E+00	8,225E+02
2	1	0,000E+00	0,000E+00	0,000E+00	-3,251E-01	-1,201E+00	0,000E+00	3,178E+01	-3,625E+02	1,781E+02	0,000E+00	0,000E+00	8,093E+02
3	1	1,500E-01	0,000E+00	3,000E-02	-6,795E-01	-7,288E-01	-6,643E-01	-6,396E+01	-8,616E+01	-5,714E+01	-1,165E+01	0,000E+00	8,243E+02
4	2	1,500E-01	0,000E+00	3,000E-02	-6,259E-01	-6,670E-01	-6,643E-01	-6,398E+01	-8,616E+01	-8,473E+01	-1,165E+01	0,000E+00	8,243E+02
5	1	0,000E+00	0,000E+00	3,000E-02	-6,412E-01	-5,252E-01	-1,009E-01	-1,407E+02	-8,849E+01	1,024E+02	0,000E+00	0,000E+00	8,139E+02
6	2	0,000E+00	0,000E+00	3,000E-02	-5,434E-01	-4,467E-01	-1,009E-01	-1,407E+02	-8,847E+01	9,825E+01	0,000E+00	0,000E+00	8,138E+02
7	2	1,500E-01	0,000E+00	7,000E-02	4,553E-01	5,807E-01	-4,733E-01	1,671E+02	2,348E+02	-3,344E+02	2,349E+00	0,000E+00	8,160E+02
8	3	1,500E-01	0,000E+00	7,000E-02	-1,129E-01	-8,777E-02	-4,733E-01	1,671E+02	2,348E+02	-8,062E+02	2,349E+00	0,000E+00	8,160E+02
9	2	0,000E+00	0,000E+00	7,000E-02	-4,569E-01	2,865E-01	-2,575E-01	-1,868E+02	2,146E+02	-7,913E+01	0,000E+00	0,000E+00	8,139E+02
10	3	0,000E+00	0,000E+00	7,000E-02	-2,023E-01	-5,359E-02	-2,575E-01	-1,868E+02	2,146E+02	-3,358E+02	0,000E+00	0,000E+00	8,139E+02
11	3	1,500E-01	0,000E+00	3,700E-01	1,884E-01	2,242E-01	-4,060E-02	2,482E+02	3,449E+02	-3,700E+02	4,061E+01	0,000E+00	6,838E+02
12	4	1,500E-01	0,000E+00	3,700E-01	-8,395E-04	2,742E-03	-4,060E-02	2,482E+02	3,449E+02	-8,254E+02	4,061E+01	0,000E+00	6,838E+02
13	3	0,000E+00	0,000E+00	3,700E-01	2,100E-01	2,428E-01	-4,370E-02	2,806E+02	3,692E+02	-4,043E+02	0,000E+00	0,000E+00	7,048E+02
14	4	0,000E+00	0,000E+00	3,700E-01	-1,791E-04	3,103E-03	-4,370E-02	2,806E+02	3,692E+02	-8,944E+02	0,000E+00	0,000E+00	7,048E+02

Table 8.1 Results from BISAR 3.0

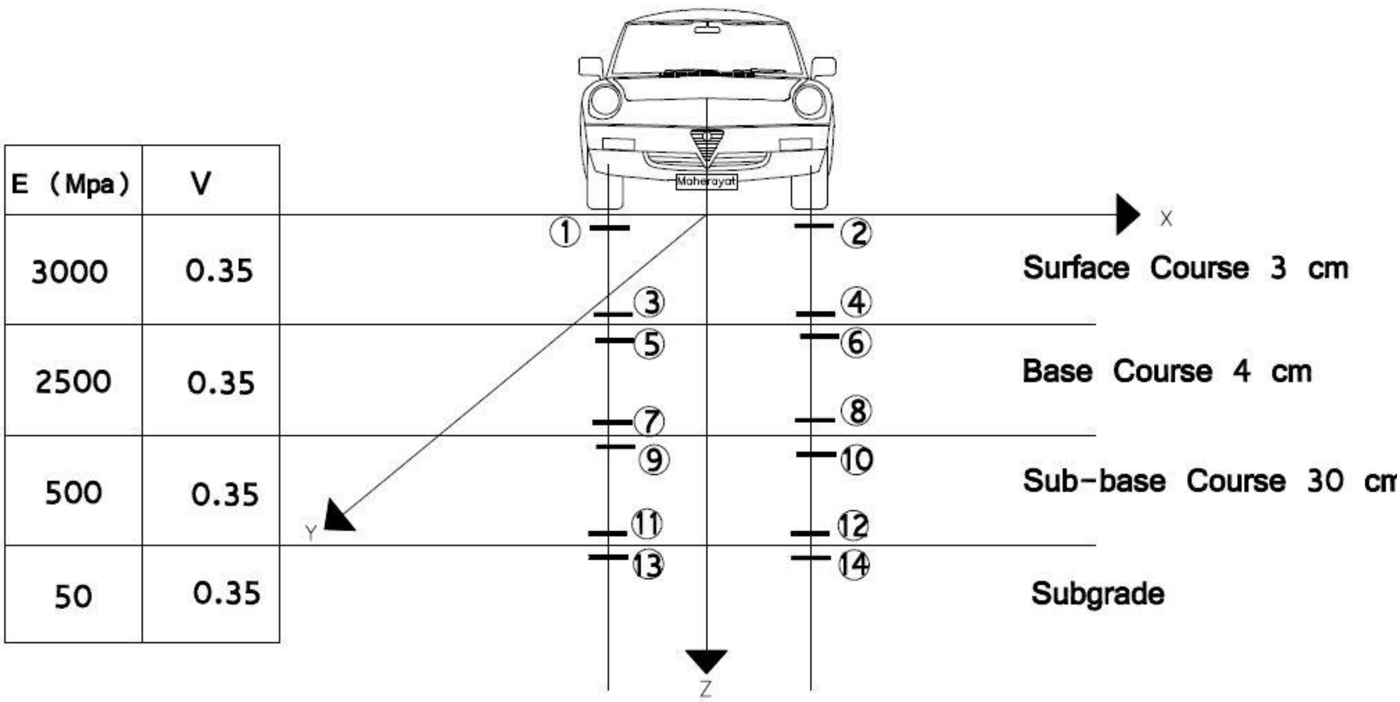


Figure 8.3: Simplified Process of Calculation

We have: Poisson's Ratio = 0.35

Modulus of Elasticity E = 3000 MPa for the Surface Layer.

= 2500 MPa for the Base Course.

= 500 MPa for the Sub-base Course.

= 50 MPa for the Subgrade.

We are, essentially focusing on: The points **1** and **2** for the Displacement value (↓Z), to the points **7** and **8** for the Strain (XX YY), and to **13** and **14** for the Strain (ZZ ZZ).

As shown in the figure 8.3 above, the points 1 and 2 are the two points directly under the contact surface. The points 7 and 8 are the points in the bottom of the base Course. And the points 13 and 14 are located on the Subgrade layer.

The sign (**-**) in the table 8.1 in the BISAR's results means a compression and the sign (**+**) means Tension.

8.3.2 Fatigue Life Study:

Point	Displacement U (↓Z)	Strain Ɛ (XX YY)	Strain Ɛ (ZZ ZZ)
1, 2	822.5 μm		
7,8		234.8 μStrain	
13,14			-894.4 μStrain

Table 8.2: BISAR results of the studied points

$$\varepsilon = K(\frac{N}{10^6})^{(-\frac{1}{a})} * (\frac{E}{E'})^{(b)} \qquad \text{(Equation 15)}$$

Where: ε = Tensile Strain

K, a and b = Constant Function of material and environmental Conditions

N = Number of Loads to cause a certain Deterioration

E = Modulus of the Material

E' = Reference Modulus

●Point 7 and 8: Bottom of the Base layer:

K = 224

a = 4,32

ε = 234,8

b = 0

After few calculation: **N = 815935,218 HVA** (Heavy Vehicle Axle)

●Point 13 and 14:

K = 885

a = 4

ε = -894,4

b = 0

After few calculation: **N = 958618,752 HVA** (Heavy Vehicle Axle)

→ Thus, the results are not bad, but also, not so good. In fact, the results are acceptable and we are in a **medium level** road. However, that can represent a problem in the future, since from the traffic volume forecast, it will be a higher traffic volume and the road characteristics will not be sufficient. So, we suggest to improve the road quality (by improving layers characteristics) or to use other types of pavement such as the **Perpetual Pavement**.

8.3.3 Design Recommendations: Perpetual Pavement:

Perpetual Pavement is a term used to describe a long-lasting structural design, construction and maintenance concept. A perpetual pavement can last 50 years or more if properly maintained and rehabilitated.

The perpetual pavement consists of thick asphalt over a strong foundation design with three HMA layers, each one tailored to resist specific stresses:

***-HMA base layer*:** This is the bottom layer designed specifically to resist fatigue cracking. Two approaches can be used to resist fatigue cracking in the base layer. First, the total pavement thickness can be made great enough such that the tensile strain at the bottom of the base layer is insignificant. Alternatively, the HMA base layer could be made using an extra-flexible HMA. This can be most easily accomplished by increasing the asphalt content. Combinations of the previous two approaches also work.

-Intermediate layer: This is the middle layer designed specifically to carry most of the traffic load. Therefore it must be stable (able to resist rutting) as well as durable. Stability can best be provided by using stone-on-stone contact in the coarse aggregate and using a binder with the appropriate high-temperature grading.

-Wearing surface: This is the top layer designed specifically to resist surface-initiated distresses such as top-down cracking and rutting

⟶In order to work, the above pavement structure must be built on a solid foundation.

Figure 8.4 below shows an example cross-section of a perpetual pavement design:

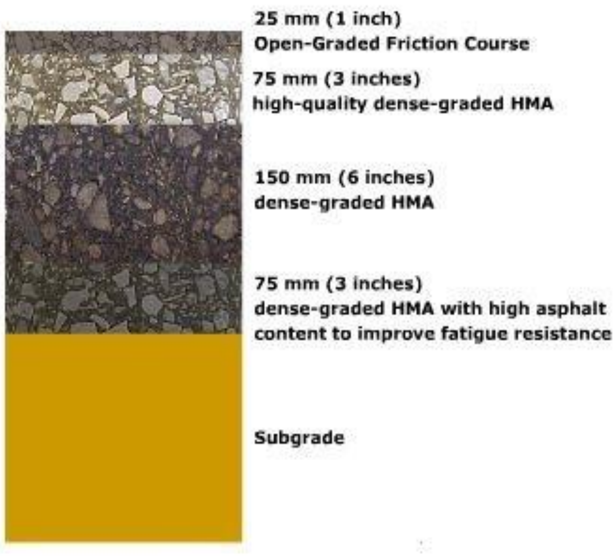


Figure 8.4: Example of a perpetual Pavement Design

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9 Conclusions:

Basically, all hard surfaced pavement types can be categorized into two groups, flexible and rigid. Flexible pavements are those which are surfaced with bituminous (or asphalt) materials. These types of pavements are called "flexible" since the total pavement structure "bends" or "deflects" due to traffic loads. A flexible pavement structure is generally composed of several layers of materials which can accommodate this "flexing". On the other hand, rigid pavements are composed of a PCC surface course. Such pavements are substantially "stiffer" than flexible pavements due to the high modulus of elasticity of the PCC material. Further, these pavements can have reinforcing steel.

Maintenance is an essential practice in providing for the long-term performance and the esthetic appearance of asphalt pavements. The purpose of pavement maintenance is to correct deficiencies caused by distresses and to protect the pavement from further damage. Pavement maintenance can be divided into Preventive maintenance and Structural maintenance.

Asphalt pavement recycling is the recycling or reusing an existing asphalt pavement into a new and structurally sound asphalt pavement. The four common methods used in asphalt pavement recycling are the Cold in-place recycling, hot in-place recycling, full Depth Reclamation and the Hot Mix Recycling.

The horizontal alignment of a road consists of straight lines, circular curves and spiral curves, whose radius changes regularly to allow for a gradual transfer between adjacent road segments with different curve radii. However, The Vertical Alignment consists of straight segments (Leveled or Inclined) connected by sag or crest curves.

The project subject of this thesis is a road of almost 6 Km of length in the province of Bologna. After making an alignment design, a safety study of vertical and horizontal alignment is made in order to check the Sight Distance and the Stopping Sight Distance. The main risk is connected to hand turn curves where the field of vision is reduced and therefore, the safe sight distance. Fortunately, our road is made in crops and there are no big obstacles or buildings that prevent the vision. For this reason, we didn’t change any curve radius or design speed throughout the road. Moreover, a pavement design is made for this road using the program BISAR 3.0 and the results are, generally, acceptable. However, the fatigue life study showed that the road’s characteristics are not sufficient for future traffic volumes and a perpetual pavement can be a solution for such cases.

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