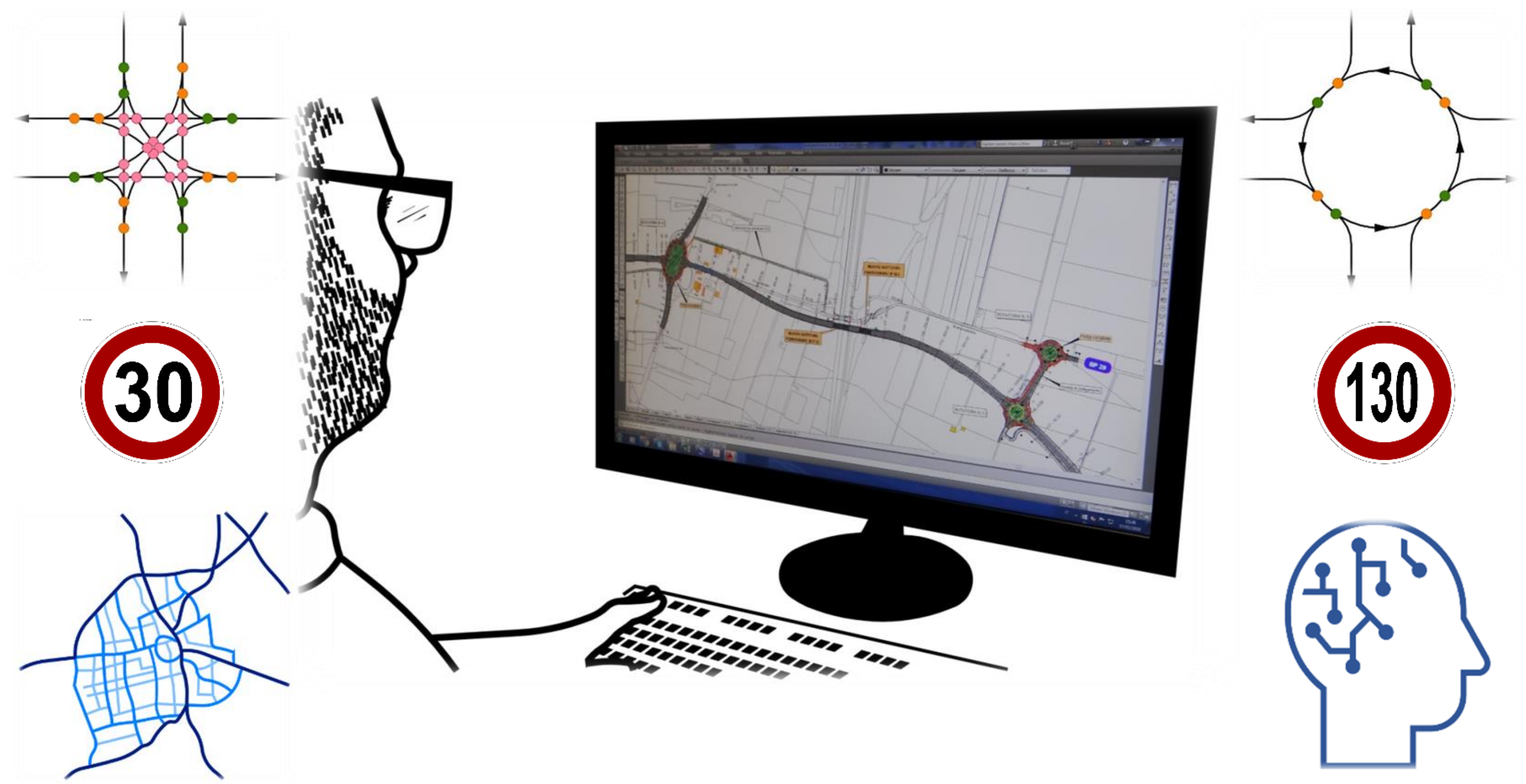
Politecnico di Bari

DIPARTIMENTO DI INGEGNERIA CIVILE, AMBIENTALE, DEL TERRITORIO, EDILE E DI CHIMICA

Pasquale Colonna
Nicola Berloco   Paolo Intini   Vittorio Ranieri

Road Safety

Technical solutions to a behavioural and technological problem with a scientific approach



FrancoAngeli
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Preface

by Pasquale Colonna

Why, as Resarchers and Practitioners, we work on Road Safety?

For a number which is included in a statistics, for an Impersonal Humanity who we will never know.

However, this IS NOT SUFFICIENT!

The "WE" of our Impersonal Humanity is actually made of several "I", of several "YOU".

In fact, one of the most determined supporter of the prevalence of the Impersonal Humanity, Karl Marx, sent the following text to his wife, on the 21st of July, 1856:

"*I can feel to be a man again, because I feel a great passion, and the variety of things in which the study and the modern culture trap us, and the scepticism which leads us to critic all the subjective and objective impressions, are deliberately made to make us smaller, weaker, mournful and irresolute. However, neither the Feuerbach's love for the human being, nor the Moleschott's love for the metabolism, nor the love for the working class, but the LOVE for the BELOVED person, FOR YOU, MAKES A MAN A MAN AGAIN.*"

Hence, I would like to dedicate this work to all the "YOU" who were known by each of us and who lost their lives in a traffic crash, each of us could dedicate this book to a known person.

I, in particular, dedicate this book to Enzo Piccinini, a great surgeon in Modena, a great academic professor and a great educator, the man who teached me that it is possible to live:

- always being themselves;
- always seeking happiness.

Enzo died on the "Autostrada del Sole" freeway near Fidenza, in his car which burned out after having tear down the safety barrier and which stopped at the base of the scarp on the 26th of May, 1999.

The following day I started to conduct research on road safety, not only because it was part of my job, but for one possible "YOU".

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Each of us should then undertake the task of giving back the opportunity of the gift of life to people, of being themselves and of being happy, to all the “YOU” that, even unsconsciously, will avoid a traffic crash thanks to the methods that this book illustrates.

Someone will thank us.

1. Introduction

All countries with a wide road network are pointing out their investments on public infrastructures, improving the existing ones rather than building new ones. The improvement of an existing road is essentially related to the improvement of road safety performances.

Nonetheless for years, there has been a consistent lack of precise knowledge about the road safety interventions and procedures as well as a lack in the protocols to be applied in practice. In recent years, in the field of road safety, the research is particularly active, and it is going forward with outstanding findings, even if it is far from the identification of unique theoretical models which explains all the experimental findings in different contexts. Maybe, this delay is due to the interaction between technical-engineering aspects and human behaviour which is almost unpredictable.

The milestone in road safety procedures has been the *Highway Safety Manual* (2010)[1], effectively the first organic work attempting to give a scientific shape to the uncertainties about the engineering approach to such a field. The importance of this manual made aware the scientific world of the necessity of better defining and unifying the knowledge acquired over the years in road safety. This need is evident, since at both the national and international level (except for the USA) a unique operative technical and regulatory protocol to design and realize safety interventions on existing infrastructures is still not provided.

The aim of the present book, which is an updated edition of the book published in 2016 in Italian language *Sicurezza stradale – Un approccio scientifico a un problema tecnico e comportamentale*[2] written by the same authors, is to be a clear and manageable manual for engineers, stakeholders and decision makers approaching to road safety issues. This edition is not merely a translation of the Italian book, but it contains several updates, especially concerning the application of the proposed method (for instance, the urban case application is completely new). This version was strongly aimed to be an open access edition, in order to disseminate as much as possible the cutting-edge methods in road safety engineering. In fact, it is aimed at prioritising the human value thanks to the benefits from reduced severe crashes, possibly provided by the guidance of this book. Hence, it has a clear academic and educational purpose, being not intended for commercial purposes.

Given the widespread composition of the audience, this book is composed of two macro sections:

- the theoretical section which aims at providing the readers with the bases of road safety, explaining which are the main topics and tools to use, shaping also a protocol for intervention on existing roads;
- the practical section which provides examples of application of the suggested protocols to deal with road safety interventions and designs. This practical section does not go deeply into the theoretical parts assuming that, the theoretical chapters would be clarifying.

This book, based on these two different sections would be a contribution in the road safety field, focusing also in detail on the Italian case thanks to a methodology which may be used in other countries outside the USA.

[1] AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

[2] Colonna P., Berloco N., Intini P., Ranieri V. (2016), *Sicurezza stradale. Un approccio scientifico a un problema tecnico e comportamentale*, Wip Edizioni, Bari, Italy.

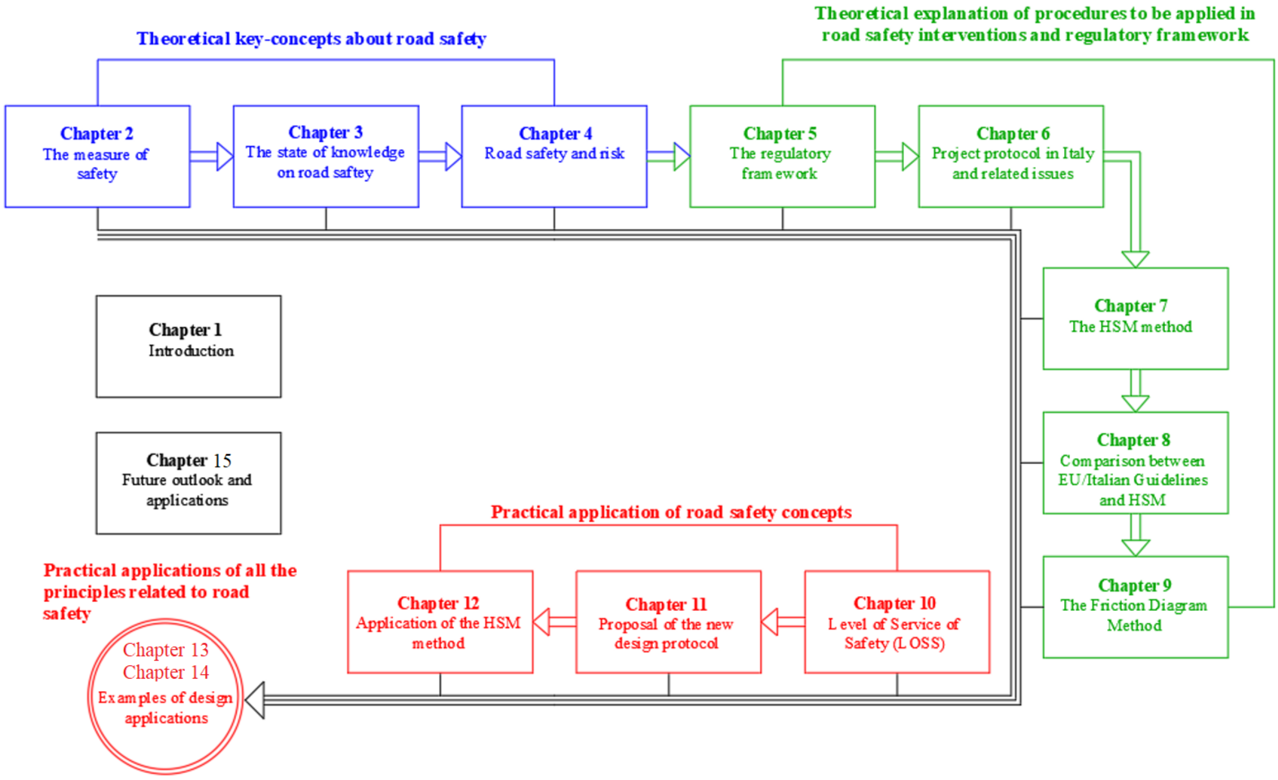

Fig. 1.1: Scheme of the book structure.

The first chapters deal with the measure of Road Safety (Chapter 2), the description of the main theories about crashes and behavioural models (Chapter 3) and the basic concepts of crash and risk (Chapter 4) dealing particularly with the importance of familiar and unfamiliar users. Italian and European Standards, Regulations (updated up to 2019) are presented in the Chapter 5. Chapter 6 comments on the critical issues of regulatory standards. In Chapter 7, a short summary highlighting the most important stages of the methodology stated by the Highway Safety Manual (HSM), currently used in the USA, is presented. In Chapter 8, this methodology is compared to the Italian Guidelines. A new methodology developed for solving friction-related safety issues is explained relying on the definition of the Friction Diagram Method, FDM (Chapter 9). However, this method is currently under validation and then its explanation is shown here but, it could also be overlooked while applying the proposed integrated framework by practitioners. The explanation of one of the methods used to identify the crash risk, developed and mostly used in the USA for performing safety diagnosis aimed at reducing crashes: the Level of Service of Safety (LOSS), is shown in Chapter 10.

A new protocol to design the safety interventions on rural roads (Chapter 11) is provided as well as the calibration of existing SPFs and estimation of new local SPFs according to the HSM method for both the rural and urban contexts in Italy (Chapter 12). Two Italian applications of this protocol are described as examples, one dealing with the rural context (Chapter 13), the other one dealing with road segments and intersections in the urban context (Chapter 14). However, unavoidably, this framework keeps some answers unquestioned and open new scenarios about road safety concerns, so future outlooks and applications are suggested at the end of this book, in Chapter 15.

2. The measure of safety

The *stake-holders* of road crashes, namely all those who in some way can be involved in the process that leads to the crash, are various. The main ones are the following:

- the users of the automotive transport:
    - drivers (characterized by several behavioural variables, according to the year when the license has been got or to a different risk aversion);
    - passengers;
- users of other modes of transport;
- vehicle fleet;
- road Network;
- external environment.

Road Safety (and therefore crashes) can be measured by several indicators.

The variables which affect road safety analysis are many and complex. Therefore, the trend highlighted by different metrics could be non-homogeneous or even charachterized by an opposite trend.

The comparison of data from the safety analysis should be performed as follows:

- splitting the effects related to each variable,
- evaluating the most suitable metric for the stakeholders and for the risk factors involved in the analysis.

2.1 Criteria for measuring road crashes

It is blatant that measuring Road Crashes requires clear goals about the aim of the Measure:

- which are the benchmark stake-holders;
- which are the benchmark risk factors.

Crashes can be also evaluated by crash frequencies, selected by type and severity. Crash frequencies can be measured as:

- number of minor crashes/year;
- number of serious (with injuries) crashes/year;
- number of fatal crashes/year;
- number of single vehicle crashes/year;
- number of vehicle-pedestrian crashes/year;
- number of crashes with all the other possible combination.

In some cases, knowing and comparing crashes occurring in different locations, intended as administrative or geographical areas, could be interesting. In such case crashes within the areas and over time are measured as crashes frequencies (number of crashes per unit of time). Sometimes crashes are also referred to the local population within the area of interest (N crashes/(year x inhabitant)).

Clearly, these indicators are affected by many other significant parameters. For this reason, making some assessments before choosing the metric should be necessary. The possible assessments to be conducted are listed below:

- efficiency and size evaluation of the vehicles fleet (between different areas and over time):

    - it might be accurate to evaluate crash tendencies according to the fleet quantity and quality metrics (e.g. new cars/cars on the road);
- efficiency and evaluation of the number of drivers (between different areas and over time):
    - it might be appropriate to evaluate crash tendencies according to the owning time of driving license and the number of driven kilometres;
- assessment of drivers' risk aversion (on the total number of crashes or on particular crash categories);
- evaluation of the equipment and efficiency of the infrastructure network (between different areas and over time):
    - crashes within an area are affected by quantity and type of road infrastructures (km of lanes per person or km of lanes per squared km or km of lanes used by the traffic in a set time interval, i.e. km of lanes per travelled km in a set time interval).

Once all these considerations have been made, the desired comparisons could be performed with the aim to be as homogeneous as possible.

Some examples of comparisons are given below:
- safety comparison (crashes selected by type and severity) between two different zones;
- safety comparison between two different road segments in the same zone;
- safety comparison between two different segments of the same road;
- safety comparison for the same road segment by making the traffic vary;
- safety comparison for the same road segment by making another variable vary (geometry, pavement, equipment and road signs, standards);
- safety comparison for the same road segment by making the time variable vary (assuming that all other variables are constant or explaining the variations observed after modifying other variables);
- safety comparison for the same area versus time (assuming the other variables constant, or explaining the observed variations);
- comparison between roads with different capacity;
- comparison between roads with different traffic flows.

However, road safety practitionners should use, as a metric, the expected mean crash frequency (number of crashes/year), ceteris paribus (or under particular fixed conditions).

The crash rate is another widespread metric, that takes into account the exposure to risk and therefore the traffic volumes affecting the road. The meaning of the crash rate is explained below.

The crash rate [Number of crashes/(vehicles x km)] is defined as the likelihood to be involved in a crash that a generic vehicle shares with all the vehicles running along a certain road segment. The road segment in question is characterized by a crash frequency and the vehicle likelihood of being involved in a crash (while travelling along that segment) is given by the expected crash frequency. This likelihood is shared by all the vehicles travelling on that road segment over the fixed time interval. All the variables are assumed to remain constant.

In order to appropriately choose the metric, the following five considerations should be considered.

1) The size of the system investigated

The choice of the size of the system is important in general for the crash measurements and, in particular, when a countermeasure is implemented. The choice should be referred to the system significantly affected by the investigated phenomenon. This assessment, obviously, could be rather subjective.

2) The crashes are influenced by the population mobility

Several experiments and their statistical elaborations showed that:
- the crash rate varies with mobility per person (driven kilometres in a year)[1];
- mobility per person varies with incomes (and the economy)[2].

3) Road safety and economic conditions are related

These parameters cannot be ignored, especially while making comparisons.

[1]Wilde G. J. S. (1994), *Target Risk*, PDE Publications, Toronto, Canada.

[2]Schafer A., Victor D. G. (2000), "The future mobility of the world population", *Transportation Research Part A: Policy and Practice, 34,* 3. 171-205.

4) Alterations to system variables

Any change of a variable belonging to the system (which can affect mobility and whose effectiveness may depend on the economic conditions) may result in road safety changes, different (even with opposite sign) by both type and severity.

5) Variability of the crash rate

The SPFs (Safety Performance Functions)[3] are graphs expressing the variability of the crash frequency (along the y axis) while traffic flows vary (on the x axis) on a given road. An example of such graphs is shown in Figure 2.1.

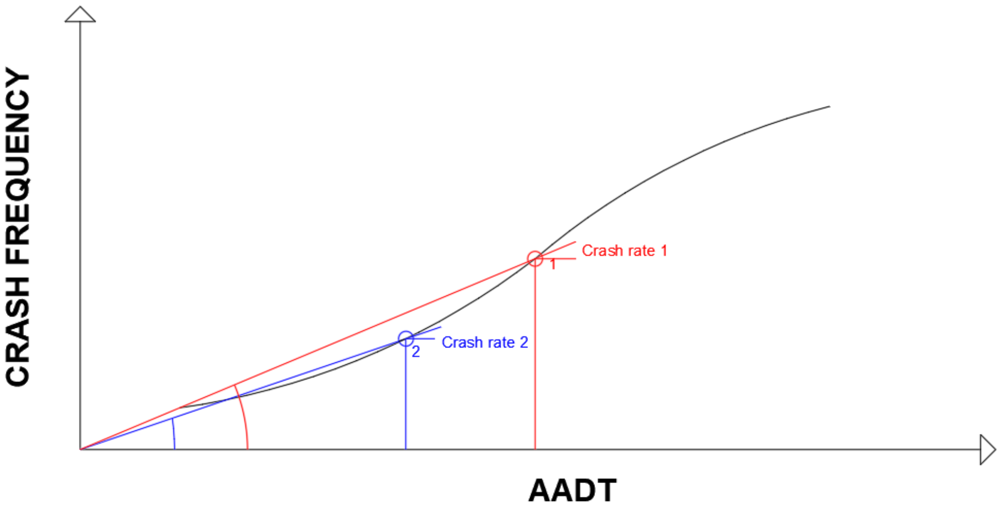

Fig. 2.1: Example of Safety Performance Function (based on Kononov and Allery, 2003[3]).

The crash rate is graphically expressed by the slope of the line from the origin of the axis to the points of the SPF.

A qualitative interpretation of the trends shown in Figure 2.1 (based on Kononov and Allery, 2003[3]) is explained as follows: the vehicle is considered as an isolated vehicle when the traffic volume is very low. In this scenario, the user behaviour, and therefore the crash probability is not affected by either the vehicles travelling in the same direction, or those travelling in the opposite direction. The crash rate (meant as aforementioned) therefore stays constant and equal to the typical value of an isolated vehicle on that road segment. A slight increase in traffic is not able to affect the user behaviour and the speed choice. Instead, the possibility of interfering with other vehicles increases in both directions (i.e., case of undivided roads). For such reason, the crash likelihood is greater than that of the isolated vehicle case. Hence, the straight-line slope that connects the origin with a point on the SPF increases. When traffic increases even more, the driver is forced to change his behaviour, making it more cautious. This effect compensates the higher probability of interfering with other vehicles. The crash rate is therefore again constant and equal to the typical value of a vehicle travelling along a road segment with a forced level of service. If the traffic increases in a remarkable way, the interactions among vehicles become prevalent (with a greater number of vehicles, the manoeuvre limitations drastically increase due to congested roads). In this case, the crash likelihood decreases as well as the straight-line slope in the figure 2.1.

Therefore, comparing sites based on the crash rate may be absolutely not-homogeneous due to the traffic variable. For this reason, a comparison based on the expected crash frequency should be preferred.

[3]Kononov J., Allery B. (2003), "Level of Service of Safety, Conceptual Blueprint and Analytical Framework", *Transportation Research Record 1840*(1), 57-66.

2.2 References

Kononov J., Allery B. (2003), “Level of service of safety: conceptual blueprint and analytical framework”, *Transportation research record*, 1840(1), 57-66.

Schafer A., Victor D. G. (2000), “The future mobility of the world population*”, Transportation Research Part A: Policy and Practice,* 34(3), 171-205.

Wilde G. J. S. (1994), *Target risk*, PDE Publication, Toronto, Canada.

3. The state of knowledge on road safety

3.1 Theories about accidents and behavioural models

3.1.1 Theories about the accident phenomenon[1]

A theory about the accident phenomenon attempts at answering to a huge number of questions:

- Why do accidents happen?
- Which are their causes?
- Are these causes able to generate effects in a probabilistic manner?

The most widespread logic behind the theories about accidents is that once the causes of an accident have been identified (at least the most important ones), the causes themselves may be corrected or solved, preventing the crash likelihood.

In the last century (obviously since the first concerns about crashes have arisen), five different theories have tried to explain accidents (summarized in the Figure 3.1).

- *Accidents as random events* (1900) Bortkiewicz (1898)[2], investigated the frequency of deaths due to horse kicks in the Prussian army through a Poisson's model, that highlighted a significant correspondence with observed data. Hence, accidents were considered as causeless phenomena, merely random, therefore they could not be neither understood nor foresee. Consequently, there was a drastic drop in the interest towards preventing/reducing accidents.
- *Accidents proneness theory* (1920). Greenwood & Yule (1920)[3] studied the accidents proneness of a group of workers in a munition factory, where an unusual concentration of accidents in a small group of workers was observed, by using a negative binomial model. It was proposed that some people, due to their physical and behavioural characteristics would be more prone to have accidents than others, an approach fostered also by the progresses in the psychology field (e.g. Freud's studies). Hence, in order to achieve drastic accident reductions, people more prone to accidents should be identified by psychophysical tests and distanced from social activities. Obviously, this approach did not lead to remarkable results.
- *Causal accident theory* (1940). A new approach to the problem came from microbiology, which tried to identify the microorganism carrying infections. Hence, it was proposed that a detailed analysis focused on the identification of the actual accident causes (through a detailed reconstruction of the event) can lead to prevent them. Findings of this study affirm that accidents are multi-causes events. The main causes identified by this study have been human (mostly), human-vehicle/road interaction, surrounding environment, and road/vehicle interaction. Therefore, following these findings, in the Fifties, a massive road educational program became crucial for road accidents prevention.
- *Systems-epidemiological accident theory* (1960). Accidents are the result of a wrong interaction among the components (drivers, vehicles, roads, environment) of a complex system. Hence, whether the Causal accident theory assesses that the human incompetence is the main cause of accidents under given conditions, the system theory searches for where the incompetence comes from, considering that the system is not adequately designed for human capabilities. Technical improvements of the transportation

[1] In this paragraph some concepts are exposed, firstly published in the book: Elvik R., Vaa T. (2004), *The Handbook of road Safety Measures*, Elsevier, Amsterdam, Nederland.

[2] Bortkiewicz L. von. (1898), *Das Gesetze der kleinen Zahlen*, B.G. Teubner, Leipzig, Deutschland.

[3] Greenwood M., Yule, G. U. (1920), "An Inquiry into the Nature of Frequency Distributions Representative of Multiple Happenings with Particular Reference to the Occurrence of Multiple Attacks of Disease or of Repeated Accidents", *Journal of the Royal Statistical Society,* 83(2), 255-279.

system components (road and vehicle design) are then proposed as a solution. Findings were more effective than those obtained with the previous theories.

- The *behavioural theory of accidents*, including the theory about the risk homeostasis (1980), which will be deepened in the following sections.

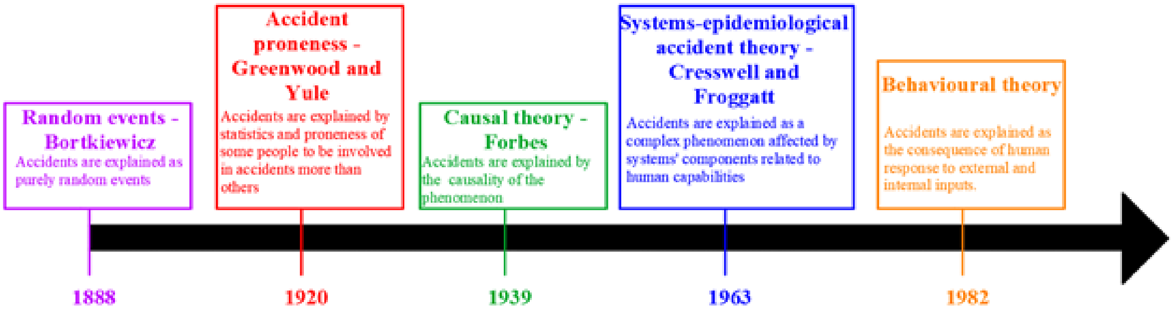

Fig. 3.1: Theories about accidents over years (based on Elvik and Vaa, 2004[1]).

Each theory presented above represents a partial point of view which describes a part of the truth, but it cannot fully explain the accidents. In fact, all the following statements are true:

- Accidents are in some way random;
- There are people more often affected by accidents than others;
- Human mistakes and errors are predominant in the accident frequency;
- Sometimes, road systems hide high complexities.

There could be a relationship among all the different theories, but it may be highly complex: some studies attempt to explain the variations in the number of accidents (statistical methods), some others attempt to generally explain the safety system. Relevant factors for one of these levels could not be relevant for other levels as well.

3.1.2 Critics to the statistical methods (Why are they not enough?)

The systematical approach to Road Safety reveals the following characteristics and problems:

- the Safety is estimated based on the expected number of accidents;
- there is no agreement on the most effective metrics;
- long-term observations are necessary (where boundary condition changes are more likely to happen);
- there is no consensus on the causes (and so on the factors to be taken into account in the models);
- the accident is a multi-cause event in a complex environment;
- the human behaviour is not considered influential with respect to accident, except for what concerns the psychophysical alterations.

Results of applications can also be very different. Sometimes the final solution is undetermined, so decreasing the reliability of using a statistical method only, in a complex system where boundary conditions always change.

How could the error be explained?

Engineering relies on math as a mean, adapting it to the reality through models.

Results from the model are approximated because they are models and not the actual reality. The validity of the result calculated by the engineer, thanks to math or statistics, depends on the approximation rate used.

The model should be applied to a close and well-defined system. Which are the boundary conditions that define if an element belongs to the model system or not? These conditions are set in order to ensure that the analyzed system can be self-sufficient. This implies that there will not be elements, likely neglected in the model, capable to significantly modify the results of the model itself.

Thus, the engineering approach to problems is probabilistic, not deterministic.

In a wide range of engineering problems, in accordance with the hypothesis, the overlapping between reality and model is very precise. Consequently, the error may be considered negligible.

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Problems related to the traffic management could, conversely have errors even equal to 25%, because of the low correspondence between model hypotheses and reality.

The most plausible hypothesis for explaining such errors is that the considered system is not closed and therefore there are some external factors, not integrated in the model, which will undermine the validity of the results.

The Canadian psychologist Gerald Wilde inquired about the causes of such errors trying to find out the mathematical abuses of engineering. His studies demonstrated the human factors' influences in road traffic. The utmost finding of Wilde's theory is that *the risk* which drivers are ready to accept is what mostly affects the accidents on roads.

This theory started from a property belonging to living organisms: *the homeostasis*. Therefore, this theory is called Theory of risk homeostasis.

3.1.3 Driving behaviour models

Among road engineering problems, road safety seems to be, undoubtedly, one of the most complex, as the wide and various studies in this field prove. Within this panorama, clear horizons of interventions are still not available.

The road engineer, whose main goal is to increase safety on a road, has still not figured out an unambiguous answer to this question.

The problem is related to some other additional questions. For example, which is the measure of safety? Is an absolute level of safety to be reached existing? How far is it possible to make safe a road segment? Which are the useful knowledge for safety? And how they could improve it? At the end, what could it be possible to do?

The lack of sure replies to the aforementioned questions has a twofold effect: on one side the research is thriving and active in this field, but on the other side the directions of the studies are multiple and divergent.

3.1.3.1 Road safety definitions

Connecting safety and crashes data is surely a very intuitive concept. Several methods propose the estimation of the mean number of expected crashes as a metric for the safety measurement[3, 4, 5, 6]. However, there are differences among them, as for instance the best function that approximates the crashes distribution or the most suitable metric to be monitored.

Is it better to use the crash frequency or the crash rate (meant as the crash frequency referred to traffic conditions)? [7, 8, 9, 10]

Nevertheless, the fundamental discrepancy among different accident predictive models lies in the assumptions about the determining crash causes, from which the models are elaborated.

A wide range of studies links the infrastructure characteristics to the estimation of crash frequency. In the current literature, one of the most accredited relationship is that represented by a negative binomial distribution. The equation which links the predicted crash frequency to infrastructure characteristics for road segments, assuming a negative binomial distribution, is the following:

[4] Hauer E. (2001), "Overdispersion in modelling accidents on road sections and in Empirical Bayes estimation", *Accident Analysis and Prevention*, 33(6), 799-808.

[5] Miaou S., Lum H. (1993), "Modeling vehicle accident and highway geometric design relationships", *Accidents Analysis and Prevention*, 25(6), 689-709.

[6] AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

[7] Pfundt K. (1969), "Three difficulties in the comparison of accident rate", *Accident Analysis and Prevention*, 1(3), 253-259.

[8] Hakkert A. S., Livneh M., Mahalel D. (1976), "Levels of safety in accident studies: a safety index", *Proceedings of the 8th Australian Road Research Board*, 5, 1-6, Victoria.

[9] Mahalel D. (1986), "A note on accident risk", *Transportation Research Record 1068*, 85-89, TRB, Washington DC, USA.

[10] Hauer E. (1995), "Exposure and accident rate", *Traffic Engineering and Control,* 36(3), 134-138.

$$E(Y) = e^{a_0} \times L^{a_1} \times V^{a_2} \times e^{\sum_{j=1}^{m} b_j \times x_j} \quad \text{(Eq. 3-1)}$$

Where:

- E(Y) = predicted crash frequency;
- L = road segment length;
- V = traffic volume of the road segment;
- x_j = each variable added to the model;
- a_0, a_1, a_2, b_j = model coefficients.

The Highway Safety Manual (2010)[6] reports different functions for predicting the crash frequency, including factors for modifying it. Such factors are defined considering the effect of the implementation of specific measures or countermeasures on the basic crash estimates determined by the model.

In order to consider systematic fluctuations, possibly neglected by the model, many studies have proposed the Empirical Bayesian approach[6, 11] , rather than pure statistic regressions.

The Empirical Bayesian model weights the results of the pure statistics regression by comparing those results with on-field detections on similar roads.

The crash rate predictive models include a wide range of cases referring to different road section types[12, 13].

However, different models could have different outcomes[11] and there is still uncertainty in the determination of the adequate metrics. For these reasons, the solutions are still undetermined. These concerns compromising the validity itself of the use of the statistic method in a complex system with always different and ever-changing boundary conditions.

Another factor of complexity is related to the observation period. A long period of observations is necessary, but at the same time the chances of external environment changes drastically increase as well as the rate of uncertainty of the measurement[14, 15, 16]. The solution indetermination rises more if the object under evaluation is a road in designing phase, extrapolated by its external context.

Other studies[17, 18, 19, 20] have so proposed surrogate measures of crashes, starting from the idea that the number of vehicles is proportional, in a certain way, to the occurrence of dangerous situations.

This concept could be represented by "Hayden pyramid"[18] (Figure 3.2). This pyramid is the base of the potential conflicts method, called TCT [21, 22, 23].

[11] Hauer E. (2000), "Accident modification functions in road safety", *Proceedings of the 28th Annual Conference of the Canadian Society for Civil Engineering*, London, Ontario, Canada.

[12] Mc Donald J. W. (1966), "Relationship between number of accident and traffic volumes at divided highway intersections", *National Research Council, Report 74,* Washington D.C., USA.

[13] Leonardi S., Pappalardo G. (2003), "Sicurezza delle Intersezioni stradali in ambito Urbano: porposta di modelli analitici previsionali del livello di incidentalità", *XIII convegno Nazionale SIIV,* Padova, Italy.

[14] Fuller R., Santos J. A. (2002), "Psycology and the highway engineer", *Human Factors for Highway Engineers*, 1-10, Edited by Fuller & Santos, Pergamon, Elsevier Science.

[15] Wilde G. J. S. (1994), *Target Risk,* PDE Publications, Toronto, Canada.

[16] Aschenbrenner M., Biehl B. (1994), "Improved safety through improved technical measures: Empirical studies regarding risk compensation processes in relation to anti-lock braking systems", *Challenges to accident prevention: The issue of risk compensation behaviour,* Styx Publications Groningen, Nederland.

[17] Tarko A. P., Davis G., Washington S. (2009), "Surrogate measures of safety", *White paper. Proceeding of the 88th Annual TRB Meeting,* Washington D.C., USA.

[18] Hayden C. (1987), "The Development of Method for Traffic Safety Evaluation: The Swedish Traffic Conflict Technique", *Bulletin Lund Institute of Technology,* 70, Lund, Sweden.

[19] Garder P. (1989), "Occurrence of evasive maneuvers prior to accidents", *Proceedings of 2nd Workshop of International Cooperation on Theories and Concepts in Traffic Safety*, 29-38, Munich, Germany.

[20] Grayson G. B. (1984), "The Malmö study. A calibration of traffic conflict techniques", *Institute for Road Safety Research* SWOV, Leidscendam.

[21] Glauz W. D., Migletz D. J. (1980), "Application of Traffic Conflict Analysis at Intersections", *Report No. NCHRP 219,* Transportation Research Boar, Washington D.C., USA.

[22] Maycock G., Summersgill I. (1994), "Methods for investigating the relationship between accidents, road user behaviour and road design standards", *Annex 4, International Research on Safety Effects of Road Design Standards*, EC Report prepared by the Dutch.

[23] Hayward J. C. (1972), "Near-Miss Determination Through Use of a Scale of Danger", *Highway Research Record 384,* 24-34, Washington D.C., USA.

Most of these studies show how the analysis of conflicts could give better safety evaluations than the ones coming from crash data (the collection period of data is also extremely limited)[24,25].

However, the relationship between conflicts and crashes is still subject of various interpretations[26].

The surrogate measures, compared to the expected crash estimates, have the merit of having widened the traditional engineering perspective from the sole infrastructure (and its components) to the whole system in its complexity. This extension has the benefit of taking into account human behaviour in the risk level analysis.

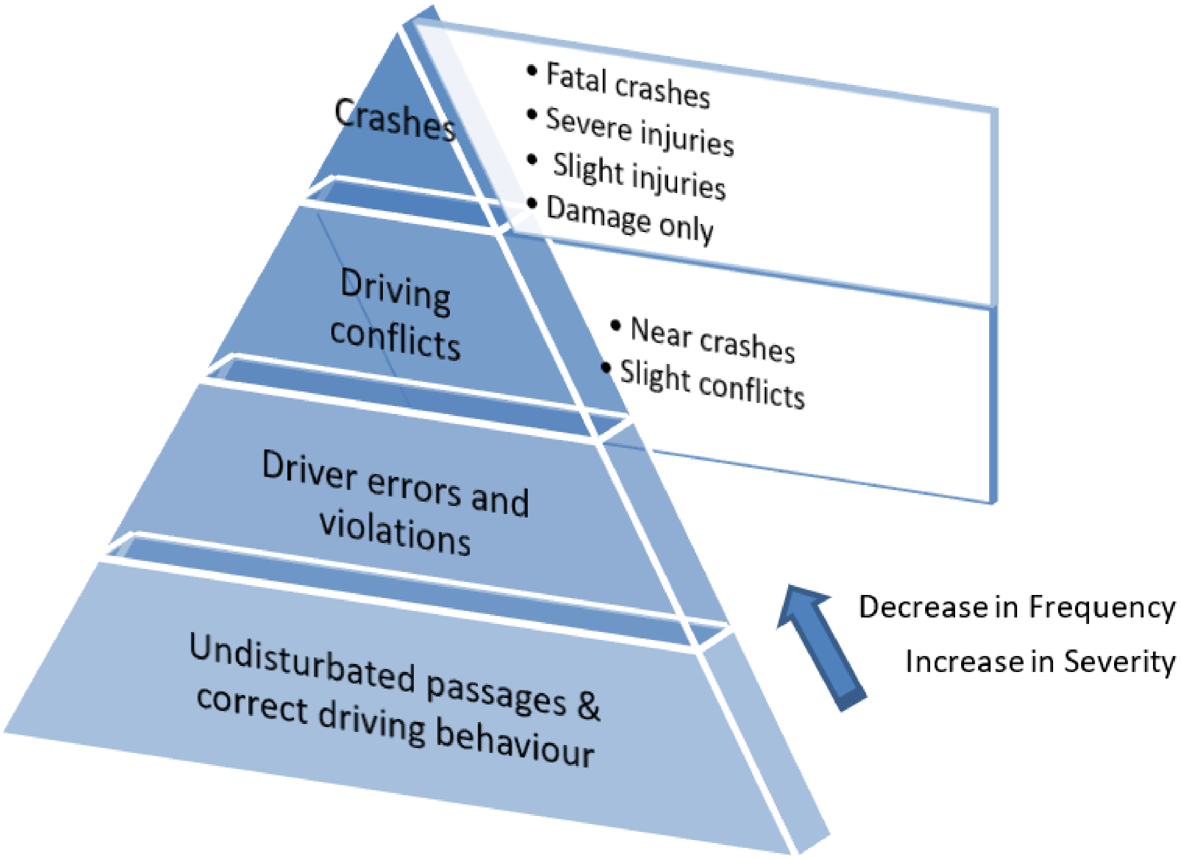

Fig. 3.2: Hayden pyramid (Based on Hayden, 1987[18]).

By the way, the behavioural interpretation in these methods is not global, as Chin and Queck (1997)[27] have highlighted.

They explained that the TCT neglected the evasive behaviours during crashes, by the drivers.

Other considerations about the ability of perceiving the road risk in an objective way were added to the previous ones, linking the crash event to a discrepancy between reality and the user's perception of it.

Several studies have further deepened the knowledge about the human factors on road safety. Among these studies, those on design consistency, which analyze the infrastructure safety related to users' responses, are particularly significant. In this way the human reaction to the road is a safety surrogate measure itself.

[24] Migletz D. J., Glauz W. D., Bauer K. M. (1985), "Relationships between Traffic Conflicts and Accidents", *Report No: FHWA/RD-84/042*, US Department of Transportation, FHA.

[25] Oppe S. (1986), "Evaluation of traffic conflict techniques", *Proceedings of the workshop Traffic Conflicts and Other Intermediate Measures in Safety Evaluation*, KTI, Budapest, Hungary.

[26] Tarko A. P. (2007), "Estimating Subjective Risk Revealed Through Driver Speed Choice", *Road Safety and Simulation International Conference RSS*, Rome, Italy.

[27] Chin H. C., Quek S. T. (1997), "Measurement of Traffic Conflicts", *Safety Science*, 26(3), 169-187.

Parameters like the effective velocity[27], psychosomatic status[28], operating speed[29, 30] or the work load[31] are introduced for estimating the safety level of the infrastructure, completely switching the focus of the observation.

The definition of safety is strictly linked to the concept of self-explaining road by the user. The target of safety is achieved by harmonizing the "problems" given by the infrastructures and the answers given by the drivers[15, 32, 33].

The basic idea is that the crash occurs when the user faces a situation which is unpredictable or not manageable for him.

Among the metrics proposed in these studies, the operating speed was mainly widespread. Several predicting models[34, 35] have been investigated for modeling the operating speed. The operating speed can reveal how the road is interpreted by the users and, in some way, it is possible to relate operating speed to the crash occurrence.

The behavioural metrics introduce another level of complexity in the debate about the road safety: the variability of the system environment – human – road over time.

The common engineering view looks at the road system seeking for the "safest possible road" with the goal of factual and invariable safety. On the other side, the behavioural metrics show different interpretations of the roads by the users. These interpretations are different from user to user, varying over time and over psychophysical status[36, 37,15].

These statements completely change the target from the road to the behaviour, inducing to think that the real aim of road safety interventions are the users with their behaviour (at the limit stage of completely disregard the road).

The study of road safety becomes necessarily multidisciplinary and assumes an empirical nature. In fact, the goal is identifying the variables which affect the human behaviour and the most suitable metrics. The final stage is finding a model able to correlate all the metrics.

The behavioural models proposed so far, although still needing to be defined and calibrated, have allowed to identify some correlations between users' choices (speed) and some of their behaviours, such as aggressiveness[38, 39].

Such models arise from tests which showed that systems implemented to increase road safety, in many cases, have proved to be less efficient than expected. In some cases, they were counter-productive, indeed, by determining crash outcomes different from the ones expected, due to an uncalculated reaction of users. Gibson and Crooks (1938)[40] provide a meaningful example, reported as follows.

Except for emergencies, more efficient brakes on an automobile will not in themselves make driving the automobile any safer. Better brakes will reduce the absolute size of the minimum stopping zone, it is true, but the driver soon learns this new zone and, since it is his field-zone ratio which remains constant, he allows only the same relative margin between field and zone as before.

[28] Taylor D. H. (1964), "Drivers' Galvanic Skin Response and the Risk of Accident", *Ergonomic*, 7 (4), 439-451.

[29] Lamm T. I. (1999), "A sectorial review of risks associated with major infrastructure projects", *International Journal of project management*, 17(2), 77-87.

[30] Lamm R., Mailaender T., Psarianos B. (1999), *Highway Design &Traffic Safety Engineering Handbook*, McGraw Hill, New York, USA.

[31] Heger R. (1995), "Driving behaviour and driver mental workload as criteria for highway geometric design quality", *International symposium on highway geometric design practices*, Boston, USA.

[32] Lamm R., Choueiri E. M., Hayward J. C., Paluri A. (1988), "Possible Design Procedure to Promote Design Consistency in Highway Geometric Design on Two-Lane Rural Roads", *Transportation Research Record, 1195*, Transportation Research Board, Washington D.C., 111-122

[33] Fitzpatrick K. (2000), *Evaluation of design consistency methods for two-lane rural highways: executive summary* (No. FHWA-RD-99-173), Federal Highway Administration, United States.

[34] Crisman B., Marchionna A., Perco P., Robba A., Roberti R. (2005), "Operating Speed Prediction Model for Two-Lane Rural Roads", *Proceedings of the 3rd International Symposium on Highway Geometric Design*, Chicago, Illinois,, USA.

[35] AASHTO (2010), *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington D.C., USA.

[36] Tarko A. P., Figueroa Medina A. M. (2006), "Implications of Risk Perception Under Assumption of Rational Driver Behaviour", *Transportation Research Record: Journal of the Transportation Research Board*, No. 1980, 8-15.

[37] Taylor D. H. (1964), "Drivers' Galvanic Skin Response and the Risk of Accident", *Ergonomics*, 7(4), 439-451.

[38] Zuckerman M. (1979), *Sensation seeking: beyond the optimal level of arousal*, Lawrence Erlbaum, Hillsdale, New Jersey, 449.

[39] Keinan G., Meir E., Gome-Nemirovsky T. (1984), "Measurement of risk taker's personality", *Psychoogical Reports*, 55, 163-167.

[40] Gibson J. J., Crooks L. E. (1938), "A Theoretical Field-analyses of automobile driving", *Am J. Psychol.* 51 (3), 453-471.

The interest in the risk perception by the user comes from the statement for which the crash probability is linked to the difference between the effective risk and the perceived risk.

Models for predicting the risk perception by user are different: they take into account the learning phase process[41], the risk level as the maximum threshold acceptable[42] or a comparison system for the desired risk level[43].

All these models, even if in different way, have in common the hypothesis for which people react to boundary condition changes in a compensative way, e.g. they respond to the perception of a safer environment with a more risky driving behaviour and vice-versa.

Nowadays, this compensation is a theme of several analysis and debates. By the way, not all the behavioural studies consider the risk as the control element. In the literature there are several models: Fuller (2000) claims that the user tries to optimize his driving commitment [44]. Vaa (2001) studied a model where the driving experience is controlled by a not-fully conscious process of optimizing the emotional status[45].

However, the behavioural approach is relatively young compared to the history of road safety. This concept provides an interesting reading-key of the road safety problem, leading to a reconsideration of engineering in this field: the design is not only the research of the safest infrastructural measures, but the behavioural control through the integration among technological innovation of informative systems, infrastructure and policies.

3.1.3.2 The behavioural aspect in the road safety estimation

Compared to the epidemiological and systemic approach, the behavioural models provide a wider and global point of view on the problem, putting together the human mobility, the social and technological environment. Behavioural models propose a relationship between crashes and human perceptions of the road. The traffic and the transportation are characteristics of a complex system and the user is just one of the components, with several and determined properties:

- the user interacts with other systems components;
- the user has a brain for thinking, he/she has problem solving capacity (not infallible), emotions and memory;
- The user expresses behaviours as subjective processes of evaluation, not always conscious.

The complexity of the interaction with the environment and the inner process has been analyzed through models which consider the problem under different facets.

The behaviour evaluation has been often relating to single aspects. A possible typological classification is reported as follows.

[41] Rasmussen J. (1987), "Reasons, causes and human error", *New Technology and human error,* Rasmussen Duncan Leplat (eds), Chichester, UK.

[42] Summala H. (1988), "Risk control is not risk adjustment: the zero-risk theory of driver behaviour and its implication", *Ergonomics,* 31, 491-506.

[43] Wilde, G. J. S. (1982), "The theory of risk homeostasis: implications for safety and health", *Risk Analysis,* 2, 209-225.

[44] Fuller R. (2000), "The task-capability interface model of the driving process", *Recherche Transports Sécurité,* 66, 47-59.

[45] Vaa T. (2001), "Cognition and emotion in driver behaviour models: Some critical viewpoints", *Proceedings of the 14th ICTCT Workshop,* 48-59, Vienna, Austria.

Tab. 3.1: Typological ranking of behavioural models.

Descriptive Models	Description of the mental processes logic	Michon (1976)[46]
Evaluations Models	Link between the driving performances and the risk perception by the user according to his/her characterization	Driver behaviour questionnaire Hazard perception test Sensation Seeking Scale Dula dangerous driving system
Interactional Models	Analysis of the interaction between different users or components, assuming outcomes	Traffic Conflicts Game - theoretic
Motivational or explicative Models	More complex models that try to explain the how and why of the mechanisms behind the behaviour	Wilde's Theory (1994)[15] Nataneen & Summala (1976)[47] Fuller (2000)[44] Vaa (2001)[45]

Although explicative models are the most exhaustive and complex models, they are still uncertain in their estimations. Therefore, each type of model proposed provides useful information for the overall analysis. Each model seems to explain some specific behavioural aspects, neglecting some others. On the other hand, some models seem to be more consistent than others.

Regardless of the type of the model, the definition and validation of model need a wide range of experiments in order to identify very complex correlations (for example, some studies rely on neural networks). The final goal is understanding the perception mechanism of the external inputs, including also the risk, its evaluation and the behavioural response.

It is necessary to define the parameters of the perceiving process and the most important behaviour metrics which may describe the response related to the relationship human-environment, as well as the mechanisms occuring when there is the switch from one to the other (human and environment): only in this way it is possible to intervene on them. With the accurate restrictions due to the specificity of each case, the human behaviour studies are focusing on what has been observed through virtual simulations[48, 49, 50].

3.1.3.3 Principal explanatory models

The development of a unique theory on the road safety needs the definition of an explanatory model.

As aforementioned, the category of explanatory models mainly includes models which link the cognitive processes and learning to rules internally produced.

Although the existence of these bounds, their relationships are still unclear because they speculate more on products of the cognitive functions (emotions, feelings, intentions and beliefs) than on the nature of the functions themselves. The main difference among the most accredited theories, on which is it possible to intervene, is constituted by their key-points, rather than their mechanisms.

The true concern of studying such theories is the awareness of the drivers' attitude or the risk perception and how this perception influences the objective safety, i.e. cognitive dynamics.

There are different models and approaches, but mostly there are different interpretations due to the complexity of the problem and the lack of shared definitions.

In the next paragraphs, the main models are described and discussed in light of personal considerations.

[46] Michon J .A. (1976), "The mutual impacts of transportation and human behaviour", *Transportation planning for a better environment*, P. Stringer and H. Wenzel (eds.), New York.

[47] Näätänen R., Summala H. (1976), *Road User Behaviour and Traffic Accidents*, North Holland/Elsevier, Amsterdam, New York.

[48] Jonah B. (1986), "Accident Risk and Risk-Taking Behaviour Among Young Drivers", *Accident Analysis and Prevention*, 18(4), 255-271.

[49] DeJoy D. (1992), "An Examination of Gender Differences in Traffic Accident Risk Perception", *Accident Analysis and Prevention*, 24(3), 237-246.

[50] Kemeny A., Panerai F. (2003), "Evaluating Perception in Driving Simulation Experiments", *Trends in Cognitive Sciences*, 7(1), 31-37.

Risk-speed compensation model

The "risk-speed compensation model" by Taylor (1964) was one of the earliest formulations of the risk compensation. Such model is based on the fact that the higher the perceived risk is, the lower the user speed will be, which means the product between risk and speed is always constant [51].

Taylor (1964)[51] run some experiments for monitoring the users' behaviours on roads, measuring at the same time the anxiety condition thanks to the Galvanic Skin Response, GSR. The GSR is measurable either as skin-conductance or as skin-resistance, Physical or emotional stresses cause an increase of the GSR, while relax is associated to a decrease of the GSR. Taylor (1964)[51] observed the relationship between this metric and different aspects of the driving phenomenon, identifying a link between the GSR average rate of the users travelling on roads and their speed on the same roads.

From the book "*Drivers' Galvanic Skin Response and the Risk of Accident*", Taylor (1964)[51]:

To summarize: a stationary observer sees that where side-turnings are closer together, accident rates are higher, speeds are lower, and also that GSR-inducing events happen to drivers more frequently. An observer in the vehicle with the subject sees that GSR-inducing events happen at a fairly constant rate wherever the driver is, may notice that the driver tends to pass side-turnings at a constant rate, and also, if he is particularly observant, that the product of the driver's average speed and the accident rate in the area where he is travelling, is approximately constant.[..]

A possible reason why the subjective risk should be distributed thus may be found in the driver's ability to vary his performance. To some extent at least, he can voluntarily influence the risks he takes (for example by accepting or not accepting opportunities to overtake, or simply by going slowly or fast). Provided that he has this control, there would usually be no reason why he should wish to engage in more risk on one part of the road than on another, and in fact he could be said to be performing a self-paced task. When it is considered that the major restriction on his speed is due to other vehicles, the drivers of whom may be expected to be behaving in much the same way, driving could be called a 'collectively self-paced' task. The pacing factor, or in servo terms the error signal, may be the GSR rate: if this is raised by larger or more frequent GSRs, a slowing of pace is called for; if there are few hazards, the pace is quickened (in order to achieve the object of travelling) until they reappear.

In this model the accepted risk level is individually determined, in part by external factors (e.g. the hurry) and in part by internal factors (age, neural mechanisms, etc.).

The Taylor model is purely descriptive, and it does not define, in detail, the internal processes involved in the compensative model. Taylor describes in the model the relationship that exists between speed choice and risk. A lack of this model, though, is the uncertainty about how the external stimulus determines the perceived risk level.

Nevertheless, Taylor's model could be considered as the starting point for further studies in the behavioural fields. Obviously, researchers' point of views and interpretations of the mechanism perception-response by the user have varied the outcomes in this field.

Naätänen and Summala (1976)[52] developed one of the most outstanding study about this mechanism, linking the behaviour to the "reason" of the behaviour itself, while the risk becomes influent only after a subjective risk threshold; Fuller (2000)[44] states that the mechanism described by Taylor matches with the "driving difficulty" faced by the user; Wilde (1982)[43] instead identified the balance point as the "desired" risk, point beyond which a compensative mechanism arises.

The risk threshold model

In 1976, Naätänen and Summala[52] provided an interpretation of Taylor theories according to which reaction to the altered state of mind of users (the increasing of GSR) happened when they overcame a threshold values of

[51] Taylor D. H. (1964), "Drivers' Galvanic Skin Response and the Risk of Accident", *Ergonomics*, 439-451.
[52] Näätänen R., Summala H. (1976), *Road User Behaviour and Traffic Accidents*, North Holland/Elsevier, Amsterdam, New York.

the perceived risk. In 1996, Summala (1996)[53] himself summarized this model as follows.

From "*Accident risk and driver behaviour*" by Summala (1996)[53]:

Naätänen and Summala [..] proposed a threshold model on the assumption that in the dynamic driving situation, drivers actually control safety margins rather than some specific risk measure, and only when the risk or fear threshold is exceeded or expected to be exceeded, does it influence behaviour.

They postulated a 'subjective risk' or 'fear monitor' which alarms and influences driver decisions when safety-margin thresholds are violated. One aspect of this approach is that with repeated confrontations, drivers adapt to situations which at first elicited a 'risk response' and drive most of the time with overlearn habitual patterns based on safety margins, with no concern for risk: hence the label 'zero-risk theory'.

One major element in the Naätänen and Summala model was the motivation module, used to explain the tendency to approach the risk threshold, which includes both motives brought from outside traffic, trip-centred motives such as hurry, and those inherent in the behaviour of human beings when in movement, such as maintaining present speed and conservation of effort. Generally speaking, drivers look for opportunities to satisfy their motives in traffic, which basically means looking for sufficient gaps and for means to maintain their desired speed.

In Naätänen and Summala (1976)[52] theory the control point is the risk threshold, which for some users is too high accordingly to their characteristics. In contrast with the Wilde's theory (1982)[43] educational, informational and strengthening campaigns are not efficient according to Naätänen and Summala (1976)[52], because the risk threshold is not influenced by such policies. A safety improvement is achieved only acting on the vehicle and the road so on the perception of safety margins. The threshold would be determined by a rational cost-benefit analysis taking into account advantages and disadvantages of driving accordingly to the motivation. The model as shown by the authors is displayed in Figure 3.3.

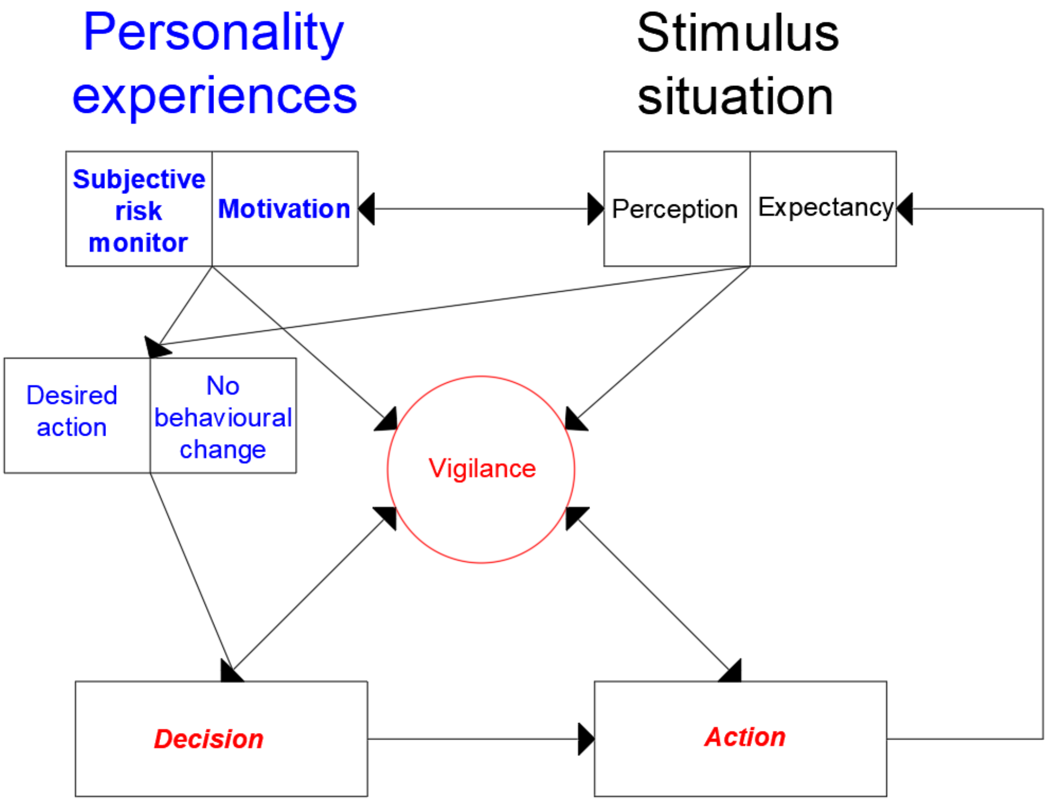

Fig. 3.3: The risk threshold model (based on Naätänen and Summala, 1976[52]).

[53] Summala H. (1996), "Accident risk and driver behaviour", *Safety Science*, 22 (1-3), 103-117.

In 1984 Fuller (1984)[54] proposed a model based on a fundamental assumption: the users act to avoid the negative consequences linked to critical situations. For this reason, they preferred to anticipate, with a minor or a greater margin, the response to this situation, according to their skills but especially to their experience.

In fact, a situation could generate some emotions which put the users in front of the choice of preventing or reducing the danger. This choice is strictly related to the drivers' behaviour and experience: the input depends on the user expectation, his motivation and his utility of anticipating or not the response. The choice of a greater margin is typical of a prudent driver who reacts deleting at the beginning the "threatening input coming from the environment". Not anticipating the response is typical of an aggressive user who doesn't care of the "threat". In the second case, if the threat does not stop, the user will decide to avoid the danger late or not to avoid it. The latter case could prevent the damage too late or in any case the crash will occur. The model proposed by Fuller is shown in Figure 3.4.

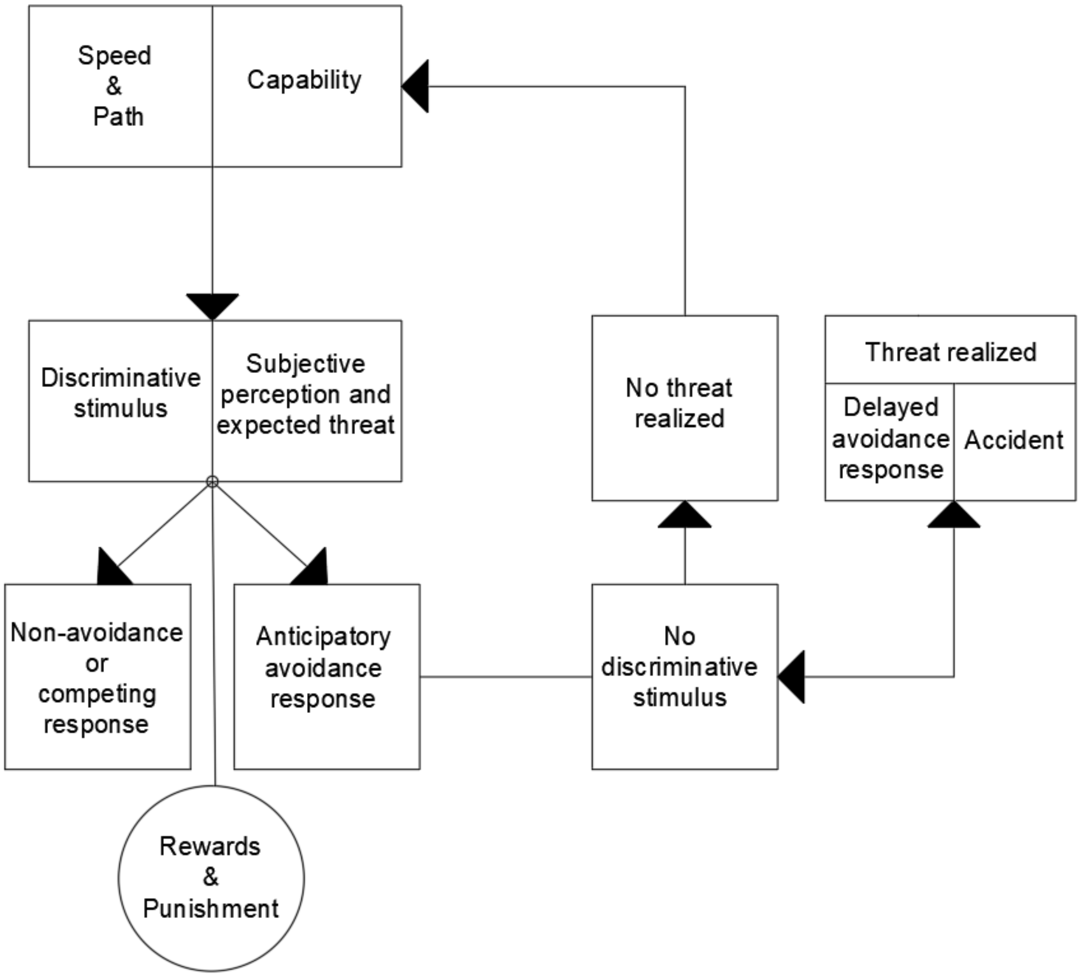

Fig. 3.4: The risk avoidance model based on Fuller R. (1984)[54].

In 2000, Fuller overcame this model[55] focusing on some thoughts about the role of the learning process and of the user experience. In 2005, Fuller explained the evolution of his perspective in the book "*Towards a general*

[54] Fuller R. (1984), "A conceptualization of driver behaviour as threat avoidance", *Ergonomics*, 27(11), 1139-1155.
[55] Fuller R. (2000), "The task-capability interface model of the driving process", *Recherche Transports Sécurité*, 66, 47-59.

theory of driver behaviour”[56]:

Once a motor vehicle begins to move, collision (or veering off the roadway) is not a matter of some refined estimate of a very low probability: it is inevitable. The probability of crashing is one, unless, of course, the driver more-or-less continuously makes direction and speed adjustments to avoid this otherwise certain outcome. For this reason, an earlier conceptualization of key elements of the driving task focused on avoidance of potential aversive consequences and the conditions for delaying an avoidance response, which had implications for safety (see Fuller, 1984[54]). In that conceptualization, objective risk of collision was assumed to be related to the extent of delay of an avoidance response, once a critical threshold had been passed. An example of a delayed avoidance response might be not slowing down when approaching a turning vehicle, which was expected to be out of the driver's path by the time it was reached. This perspective on driver behaviour was subsequently elaborated into a comprehensive behaviour-analytic model, enabling detailed consideration of the role of antecedent events and consequences in the determination of driver behaviour . In that model, subjective risk estimates were not a determinant of driver decision making, except in the profound sense of motivating the continuous avoidance of certain catastrophe, and this distinguished the approach in a fundamental way from that of the Risk Homeostasis theory of Wilde (1994[15], 2001[66]).

An equally plausible explanation of Taylor's observations as that of risk-homeostasis, however, is the proposition that drivers respond to variations in task difficulty rather than feelings of risk and that they respond to these variations both in terms of autonomic arousal and adjustments in speed.

Fuller himself proposed a new model: the subjective risk is no more the behavioural metric taken into account, but the level of driving “difficulty task” becomes the new behavioural metric. So the accent is on considering “easy” or “difficult” the task of avoiding or preventing negative consequences.

The value of the user perceived “difficulty” would be determined by an inner mechanism of comparison between the driving “commitment” and driver “capability”. This model is explained by the “task-capability” model.

From “*Towards a general theory of driver behaviour*”, Fuller (2005)[56]:

Where capability exceeds demand, the task is easy; where capability equals demand the driver is operating at the limits of his/her capability and the task is very difficult. Where demand exceeds capability, then the task is by definition just too difficult and the driver fails at the task, loss of control occurs, and this perhaps leads toa collision or the vehicle careering off the roadway. […]

At the threshold where task demand begins to exceed capability, we need not necessarily expect a sudden and catastrophic breakdown of control but rather a more fragmented degradation.

In few words, the driving difficulty is inversely proportional to the differences between the user “skills” and the “effort” due to the roadway. Skills depend on different variables: first of all, the biological characteristics of the user, as his/her ability in managing pieces of information and speed, reaction time, coordination, physical conditions; besides these, maybe the most important, the knowledge developed through learning phases and experience (road rules, procedures, ability in figuring traffic scenarios out). The whole of the described components is the up threshold of the driver competence. However, this is not always fully used because a series of human factors acts on it. Among the human factors: the attitude, the motivation, the tiredness, the distraction, the stress, the use of alcohol and drugs and the emotional status.

What about the “commitment demand” of the road? This essentially depends on a wide range of factors, as undoubtedly the road geometry and the sight conditions of the road. Moreover, there are indeed the presence of other users, the vehicle characteristics and the speed and trajectory expressed by the user. Fuller (2000)[55] considered the speed the most significative factor. The entire model is displayed in Figure 3.5.

[56] Fuller R. (2005), “Towards a general theory of driver behaviour”, *Accident Analysis and Prevention*, 37(3), 461-472.

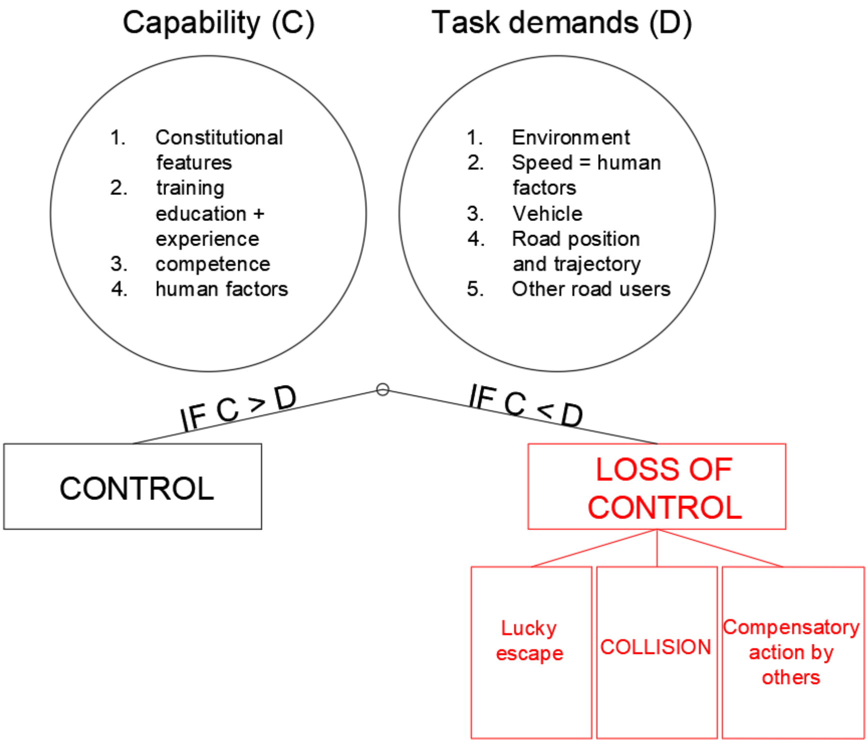

Fig. 3.5: The "task-difficulty" model based on Fuller (2000)[55].

The "Target Risk" model

Wilde, in accordance with Taylor's theory, proposes a behavioural theory in which the risk is the key element. This theory ascribes the variation of human behaviour to the compensation of the imbalance between a level of risk subjectively expected by user, based on the road perception, and his/her own acceptable risk level, which Wilde defines as "target risk"[57]. According to Wilde, the target risk is the target variable. In fact, it represents the level of risk that everyone is willing to accept[15]. Consequently, attempts to increase safety by improving road infrastructure, enhancing passive safety systems on vehicles or developing driving skills (in terms of driving skills and experience) are likely to fail because they do not address the target risk.

From *Target Risk* text, Wilde (1994)[15]:

Risk Homeostasis Theory maintains that, in any activity, people accept a certain level of subjectively estimated risk to their health, safety, and other things they value, in exchange for the benefits they hope to receive from that activity (transportation, work, eating, drinking, drug use, recreation, romance, sports or whatever). In any ongoing activity, people continuously check the amount of risk they feel they are exposed to. They compare this with the amount of risk they are willing to accept, and try to reduce any difference between the two to zero. Thus, if the level of subjectively experienced risk is lower than is felt acceptable, people tend to engage in actions that increase their exposure to risk. If, however, the level of subjectively experienced risk is higher than is acceptable, they make an attempt to exercise greater caution. In either case, people will choose their next action so that the subjectively expected amount of risk associated with that next action matches the level of risk accepted. In the process of executing that next action, perceived and accepted risk are again compared and the subsequent adjustment action is chosen in order to minimize the difference, and so forth in an ongoing manner.

Wilde points out that the compensation mechanism does not tend to keep constant the value of the risk, but homeostatic: wavering in an acceptable around of the target risk. These variations are necessary to direct the behaviour in one direction rather than in another one. Furthermore, they make the actual average constant (Wilde, 1994[15]).

[57] Wilde G. J. S. (1982), "The theory of risk homeostasis: implications for safety and health", *Risk Analysis*, 2(4), 209-225.

This apparent contradiction may be one of the reasons that the process of homeostasis is sometimes misunderstood, but it is part and parcel of its nature. A homeostatic process makes it possible to extract long-term steadiness from short-term fluctuations […]

Homeostasis is a self-correcting mechanism through its use of negative feedback.

Wilde represents the process as shown in Figure 3.6.

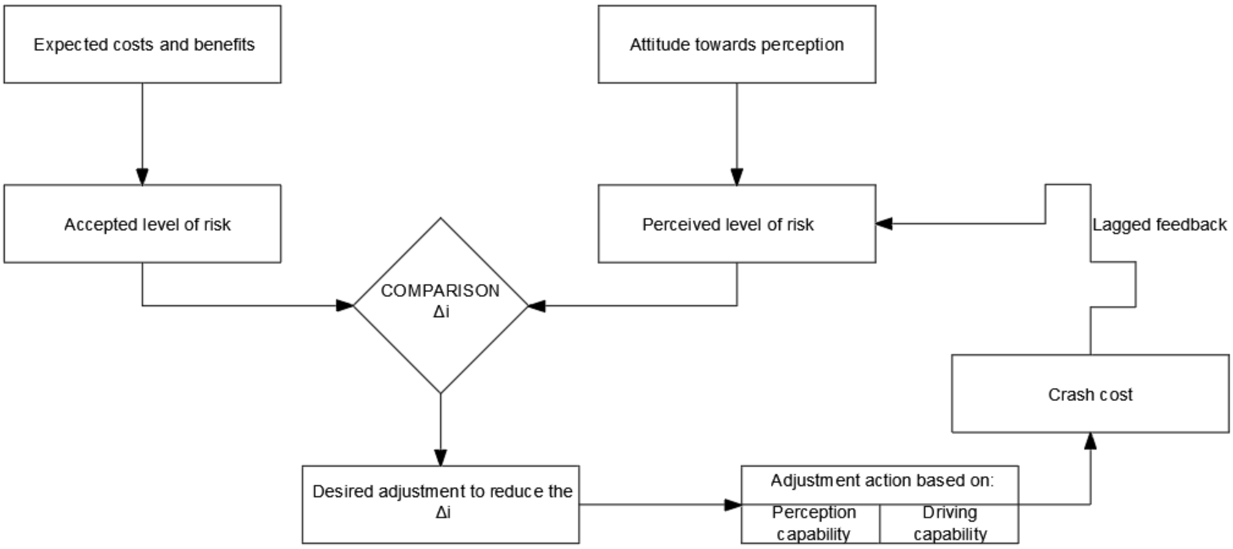

Fig. 3.6: The Homeostatic model by Wilde (1982)[57].

Wilde extends the concept of target risk to the whole society, transferring to that the contribution of everyone's actions and giving to educational and awareness-raising policies a fundamental role in reducing deaths per population unit (Wilde, 1994[15]).

Each particular adjustment action carries an objective probability of risk of accident or illness. Thus, the sum total of all adjustment actions across all members of the population over an extended period of time (say one, or several years) determines the temporal rate (i.e., per time unit of exposure to risk) of accidents and of lifestyle-dependent disease in the population.

These aggregate rates, and in particular the more direct and frequent personal experiences of danger, in turn influence the amount of risk people expect to be associated with various activities, and with particular actions in these activities, over the next period of time. They will decide on their future actions accordingly, and these actions will produce the subsequent rate of human-made mishaps. Thus, a "closed loop" is formed between past and present, and between the present and the future. And, in the long run, the human-made mishap rate essentially depends on the amount of risk people are willing to accept.

Wilde[57] demonstrates the existence of a "social" target risk thanks to the Alderson[58] study (1981) which shows a stable trend in road deaths compared to the population over the last three quarters of the 20th century, in spite of efforts made in road safety and technological innovation. Hence, Wilde linked this phenomenon to a lack of "education for life" policies aimed at reducing the target risk, through a great amount of data. Nonetheless, this model was criticized by Elvik and Vaa (2004)[59] for the uncertainty related to risk perception mechanism with respect to the road.

In accordance with Wilde's statements and homeostatic theory, though, if a risk budget exists, drivers will aim at spending their own level of risk, regardless of any engineering measures of intervention. Therefore, interventions, that raise the perception of safety, actually induce a compensatory behaviour in the user. So, the increase of objective safety must be related to measures that increase the subjectively perceived value of risk, and to educational policies to reduce the "social risk budget" (e.g. raising sanctions).

[58] Alderson M. R. (1981), *International mortality statistics*, Palgrave Macmillan, London, UK.
[59] Elvik R., Vaa T. (2004), *The Handbook of road Safety Measures*, Elsevier, Amsterdam, Nederland.

3.1.3.4 Risk compensation

In many studies different user behaviours were compared to age, gender, lifestyle, driving experience. In general, the regulation of the driving process is compensatory respect to an uncomfortable situation[60]. Users increase the speed of the vehicles according to their level of comfort[61] and vice-versa. Analyzing in particular studies on the behavioural differences between young and old people[62], between new drivers and experts[63], between men and women[64] and between users in vehicles with and without safety on-board devices[65], while cautiously considering the exclusion of other factors from the assessment, an important link is made between the user's risk perception (related to his/her own characteristics) and the compensation above described.

Therefore, the concept of risk compensation, homeostatic or not, which is still rather debated today, could be decisive in predicting and controlling the crash occurrence. Therefore Wilde (2001)[66] keeps going to update and to re-discuss the theory of risk homeostasis. For the same reason, numerous studies have again argued about this theory and tried to define the factors that get into risk assessment play.

Levels of behaviour and compensation

Michon (1976)[46] analysed the driving task of the road user and described its general logic. He identified three levels of action and control: a strategic level (planning), a tactical level (manoeuvring) and finally an operational level (control).

This representation is explained by the author himself with the Figure 3.7.

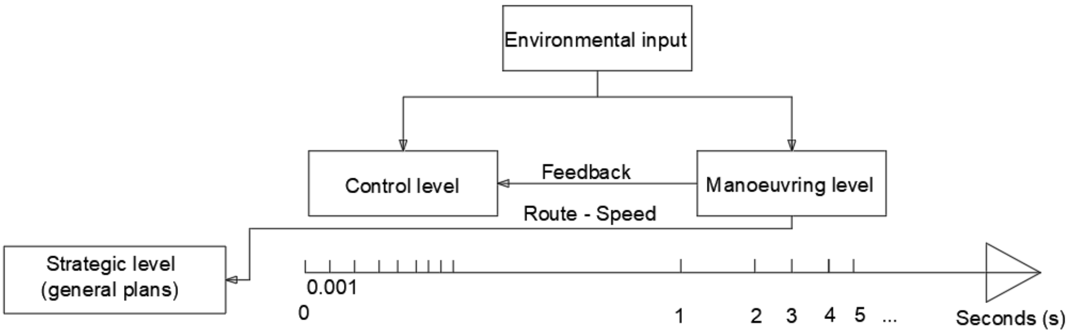

Fig. 3.7: Levels of behaviour (based on Michon, 1976[46]).

At the strategic level, the user makes decisions regarding the more general planning of his trip, from the route to the modal choice, up to the evaluation of its costs, advantages and risks.

[60] Fuller R. (2005), "Towards a general theory of driver behaviour", *Accident Analysis and Prevention*, 37(3), 461-472.

[61] Tarko A. P. (2007), "Estimating Subjective Risk Revealed Through Driver Speed Choice", *Road Safety and Simulation International Conference RSS*, Rome, Italy.

[62] Jonah B. (1986), "Accident Risk and Risk-Taking Behaviour Among Young Drivers", *Accident Analysis and Prevention*, 18(4), 255-271.

[63] Wilde G. J. S. (2005), "Risk homeostasis theory and traffic education requirements", *Proceedings of the IV ICTCT Extra Workshop "Measures to assess risk in traffic as reflected by individual test performance, in attitude measurement and by behaviour and interaction"*[Available online from: http://www. ictct. org.], Campo Grande, Brasil.

[64] DeJoy D. (1992), "An Examination of Gender Differences in Traffic Accident Risk Perception", *Accident Analysis and Prevention*, 24(3), 237-246.

[65] Streff F. M., Geller E. S. (1988), "An Experimental Test of Risk Compensation: Between-Subject versus Within-Subject Analyses", *Accident Analysis and Prevention*, 20(4), 277-287.

[66] Wilde G. J. S. (2001), *Target risk 2: a new psychology of safety and health: what works? What doesn't? And why*?, PDE Publications, Toronto, Canada.

At this level, the user's conditions, characteristics and past experiences come into play.

At the tactical level, the user trains the control maneuvers which allow him/her to deal with the various situations that arise in each circumstance during the trip. Maneuvers such as avoiding an obstacle, choosing a safe distance, turning or overtaking other vehicles are carried out according to criteria and objectives derived from the strategic level.

At the operational level, the vehicle control maneuvers, linked to the contingency, happen. They are not related to the initial route planning and occur in instants.

According to Michon (1976)[46], the interactions described in Figure 3.8 exist among the various levels and a model that includes behaviour must necessarily take this into account. A similar schematization, which identifies a level of attention and a level of reaction under the strategic level, is provided by Colonna and Berloco (2011)[67].

The schematization associate at the various levels the characteristics that determine the consequences in terms of user choice on road, starting from similar definitions.

This study claims that the user's decision-making process, at the strategic level, is aimed at choosing the risk margins within which he will move. The user will choose the value of speed reduction (ΔVi) knowing the speed limit along the route (V_{lim}) and his/her own risk aversion (Δi).

At the level of attention, the user decides to change his/her speed according to the possible change in the general external conditions that he/she initially anticipated and according to his risk aversion (Δi).

A further decrease in speed may occur whether completely unexpected events happen while driving. These can result in an instantaneous and obvious change in safety conditions, compared to the own initial driving experience, which has been recalibrated on the basis of the general conditions. The decrease in speed (ΔVe) is linked to the sudden occurrence of a certain external risk Δe and it occurs at the level of reaction behaviour (see table 3.2).

Each condition represents different levels in which the perception of risk leads to behavioural changes. Such changes are linked to a compensation phenomenon that restores the desired balance with respect to an initial situation. However, in these levels the compensation happens differently and it is based on different inputs and on non-comparable timescales.

Tab. 3.2: Typological classification of behavioural patterns.

	Strategic Behaviour	*Attention Behaviour*	*Reaction Behaviour*
Depends on	estimate of the difference between real situation and limit conditions	mutation of external conditions	unexpected events
Main characteristics	speed selection	acceleration and deceleration	braking and changing trajectory
Time of definition	medium-long	medium-short	short
Level of perception of the risk	perceived	Perceived	unperceived
Level of consciousness of the user	unconscious	conscious	conscious
Safety margin	δi (risk aversion for the uncertainty of boundary conditions)	δi (risk aversion for the uncertainty of boundary conditions)	δe (risk aversion for unexpected events)
Speed selected	Vlim – ΔVi – ΔVe	Vlim – ΔVi – ΔVe	Vlim – ΔVi – ΔVe

Risk is defined as the product between the probability of an upcoming event and the intensity of the damage caused by it:

$$R = P \times I \qquad \text{(Eq. 3-2)}$$

This risk is different at different levels: the probability considered at a strategic level derives, on a heuristic and inductive basis, from the prediction that the incident will occur (on experience and emotion basis, the user

[67] Colonna P., Berloco N. (2011), "External and internal risk of the user in road safety and the necessity for a control process", *24th World Road Congress, World Road Assosiation (PIARC),* Mexico City, Mexico.

considers a series of situations that may or may not occur under certain conditions and he/she estimates the probability); the probability at the level of reaction, on the other hand, is linked to the conditions actually existing at a given moment that induce the user to consider the probability of a given event, unexpected or not. The probability at the level of attention is similar to that of reaction but in a wider space-time panorama.

According to these considerations, the total risk is determined by the contribution of different types of risk, assessed at different levels. Each of the components must be assessed differently. In this way it is possible to intervene on them in a different way.

Therefore:

$$R = Re + Ri \qquad \text{(Eq. 3-3)}$$

Where:

- ✓ R is the risk on a road section;
- ✓ Re is the external risk: it is the difference between the risk actually existing on a road section and the risk perceived by the driver an instant before it occurs; this difference is compensated by the user with the speed reduction ΔVe. It insists on attention and reaction;
- ✓ Ri is the internal risk: it is the difference between the risk perceived unconsciously and the risk that the same user is willing to take (as assumed by Wilde[43]). The first risk acts on the behaviour and it is influenced by the experience (direct or indirect) of the user. The second risk is compensated by an attempt of obtaining a ΔVi. It insists on a strategic level.

While the perception of the external risk is linked to more or less objective situations, the perception of internal risk needs to be further deepened, trying to extend the vision beyond the pure field of road design.

According to Michon's theories (1976)[46], the user is related to the road in a complex relationship, in which several behavioural aspects coexist (Table 3.3).

However, while Michon[46] gives a hierarchical interpretation of them, the experience gained in recent neuroscience studies should lead to their unconscious and irrational interaction.

Tab. 3.3: Driving Problem Analysis Levels (based on Michon, 1976[46]).

		Behaviour level			
		1	*2*	*3*	*4*
Problem	Solver (Human quality)	Road user	Transportation consumer	Social agent	Psycho-biological organism
	To be solved	Vehicle control	Trip Making	Communication	Satisfaction of basic needs
Task	Environment	Road user	Topographical structure	Socio-economic structure	Environment
	Aids	Vehicles, Signs	Transport mode	Transportation system	Technology experience

Psychological risk perception, cognitive heuristics, experience and compensation

The risk perception depends on many factors, often specific to each situation. However, researchers are wondering the general criteria for the determination and evolution of such a perception.

Wilde (1994)[15] and also Zuckerman (1979)[68] show a strict relationship between the risk perception on roads and in other life areas. In the literature, several studies on risk perception refer to the road case because it is the most direct one in terms of perception and reaction by humans and because it is the consequence of a deliberate action.

The analysis run by the road user seen as a "social and psychological entity" (as depicted by Michon, 1976[46]) must necessarily starts from research in a more general field.

The research on risk perception arises from the assumption that the human behaviour in risky situations is just

[68] Zuckerman M. (1979), *Sensation seeking: beyond the optimal level of arousal*, Lawrence Erlbaum, Hillsdale, NJ, USA.

a decision taken in conditions of uncertainties. This uncertainty is associated with the impossibility of elaborating all the information in the same moment or with the lack of parts of information useful to come up with a sure result.

Psychometric studies investigated the process thanks to which human mind processes the information aiming at choosing the adapt behaviour in uncertain situations. These studies have become relevant in the field of risk perception because they shape the human choice in uncertain conditions as a process of elaboration of statistical information.

In these studies, the fundamental idea is that, dealing with statistical inference problems or problems related to the judgement in situation of uncertainty, people tend to rely on "cognitive heuristics", namely "shortcuts". This approach often provides a good approximation rate to the correct response, but they could lead to significative errors (Tversky & Kanheman, 1974)[69].

The different perception of risky situations (variable from subject to subject or for the same subject changing in different times) and the judgment and final response inaccuracy come from here.

Cognitive heuristics

Hume (1739)[70], in the first half of the XVIII century, already introduced the inductive process among the main human knowledge and so behavioural knowledge: human fights the not-knowledge of the future, since he is used to the familiar situations, extending to it the knowledge of the past.

From the book "*Treaties of human nature*" by Hume (1739)[70]:

From a second kind of experience I conclude that the belief that comes with the present impression and is produced by a number of past impressions and pairs of events, arises immediately without any new operation of the reason or imagination.

This habit is a biologically adapting device without which humans would not live. In everyday life, in fact, having full information is rare and sometimes there is not a correct solution to certain problems. In order to be able to make choices without certain answers, humans use heuristics of different nature, i.e. procedures which simplify the choice basing on induced pieces of information.

Nisbett and Ross (1980)[71] refer to our relatively limited capability of analyzing different parts of an information in the same moment to explain why we are vulnerable to the heuristics influence. This means that we have the need of being selective. Nevertheless, the advantage of heuristics consists in providing good enough information to a fraction of the "cognitive cost"[72].

Currently, the research attempts to show how heuristics are more suitable to help the human decisional process in naturalistic contexts[73].

Among the well-known heuristics, the "availability heuristics" are the most common. The "availability heuristics" means the tendency to assume as most likely events the ones easiest to be brought to mind.

Other common heuristics are the "emotional heuristics", as defined by Slovic et al. (2002)[74], i.e. the tendency of people to rely on the own emotional reaction when formulating a judgement or a preference.

Moreover, Slovic (1987) thought that the risk perception is highly influenced by the heuristics which contribute to reconstruct the decisional scenery in an uncertain situation[75].

[69] Tversky A., Kahneman D. (1974), "Judgment under uncertainty: Heuristics and biases", *Science*, 185(4157), 1124-1131.

[70] Hume D. (1739), *Treatise of human nature*, Penguin, London, UK.

[71] Nisbett R. E., Ross L. (1980), *Human inference: Strategies and shortcomings of social judgment*, J: Prentice-Hall, Englewood Cliffs, USA.

[72] Hogarth R. M. (1981), "Beyond discrete biases: Functional and dysfunctional aspects of judgmental heuristics", *Psychological Bulletin*, 90(2), 197-217.

[73] Gilovich T. (2002), *Heuristics and biases: The psychology of intuitive judgment*, D. Griffin & D. Kahneman, Cambridge University Press, New York, USA.

[74] Slovic P., Finucane M., Peters E., MacGregor D. G. (2002), "The affect heuristic". *Heuristics and biases: The psychology of intuitive judgment*, D. Griffin & D. Kahneman, Cambridge University Press, New York, USA.

[75] Slovic P. (1987), "Perception of risk", *Science*, 236(4799), 280-285.

The introduction of cognitive heuristics allows to understand how the risk perception is not only determined by the specific situation itself as it emerges, but by a long series of other information necessary to the behavioural choice. Such pieces of information are drawn by the former experiences, trained through an unconscious process.

The mechanism of the induction requires multiple knowledges coming from several previous similar experiences (grouped by associative processes in human mind), even if for some people the induction apparently appears after only one event.

A strict bound between learning and risk is still not sure, except for the cases when humans deal with the "reinforcement learning" or experiential learning, i.e. something coming from the consequences of the own actions rather than from what has been learnt by other experiences.

The experiment proposed by March (1996)[76] shows efficiently such statement. March used a virtual simulation to demonstrate his thesis: the aversion to risk in favor of any gain could be a direct product of the learning experience rather than of whatever personality aspect or conscious process of a superior order.

The learning system predicted two classes of objects: the "safe" option, generating a moderate prize, the hazardous "option", sometimes generating a higher income, sometimes not generate anything at all. The learning system should choose whether "approaching to" or "avoiding" objects accordingly to the own expectations developed by time. At the end of the experiment, the learning algorithm showed that only approaching to a single object, the expectations about that object-value changed. It is a process based on experiences.

The observation that the perceived risk is affected by more or less big mistakes is linked to the width of the personal experience. Such mistake could be corrected when as much as possible situations occurred, including the negative ones.

The Signal Detection Theory (Swets,1973)[77] hypothesized that the bound of the risk learning (and its perception) with the experience could be summarized in categories. The combination of bad valence and avoidance as action lead to "true positive", i.e. a correct avoidance of the risk but without feedbacks. If the action related to bad valence was the approach, the outcome was a "False-negative": the approach will lead the punishment, so it is reduced. In case of good valence with avoidance actions, there was a "False-positive", where negative behaviours persisted even in absence of feedbacks. Instead, the approach in a case of good valence is strengthened in sight of a reward. Summarizing: there is a lack of learning, in case of avoidance, whilst in case of approaching, the learning improves.

The learning process, then, follows the scheme shown in the figure 3.8, but with incorrect feedbacks this learning process is obviously altered.

If the risk in a real dangerous situation is not evaluated for what it is (false negative) and there is no damage, the "wrong" feedback would lead to mistakes in further similar situations. The personal experience which influences the risk perception is not only direct, although the direct ones generates for sure learning. The social nature of the user influences his experience in terms of indirect experience. A scheme of these dynamics and how they enter in the human behaviour is well-represented in one study about the social perception of the risk by Eiser[78] (2004), through the scheme in Figure 3.8.

[76] March J. G. (1996), "Learning to be risk averse", *Psychological Review*, 103(2), 309-319.
[77] Swets J. A. (1973), "The receiver operating characteristic in psychology", *Science*, 182, 990-1000.
[78] Eiser J. R. (2004), "Public perception of risk", *Report Prepared For Foresight Office Of Science And Technology*, University of Sheffield, UK

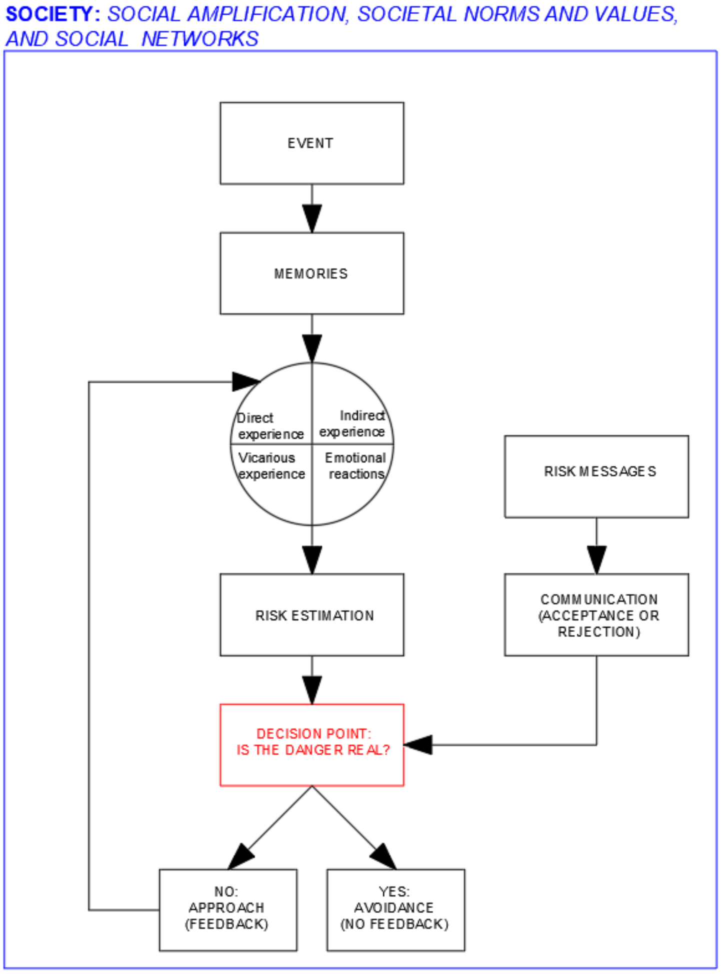

Fig. 3.8: Flow chart of the process of judgment on the presence or absence of the risk (based on Eiser, 2004[78]).

Heuristics, experience and compensation

According to the description above, on one side, there would be a regulation of behaviour in the medium-short time term. In this case, the risk and its perception are closely linked to the road conditions; the user's abilities and physical status are linked to events that change suddenly. On the other side, a regulation mechanism would also act in the long-term, or on a large scale, involving the general feeling of people, the personal tendency or aversion to risk, the judgements deriving from experience.

The compensation mechanism, studied by Taylor (1964)[37] in his speed-risk model, describes the phenomenon of speed adaptation basing on perception. Instead, the compensation described by Wilde (1982)[43] occurs in terms of general behavioural regulation, at internal and unconscious level of the person, i.e. the one that involves the perception of a real risk as actually existing (internalization) rather than not. The latter would be linked to learning the probability of the occurrence of a negative event, which has effects on the selected speed at a strategic level.

The total risk on an infrastructure is not only related to its geometric characteristics, but also to the ability to recall or not experiences that induce the user to be cautious.

Moreover, the problem, poorly dealt with in literature, of familiar road users who are subject to an experiential learning process becomes challenging and interesting. This process, if characterized by "lucky" feedback (the absence of a negative event even in correspondence with an underestimation of the risk) can generate habits and can influence the perception of risk on that road section.

A very high number of familiar users can be affected by this error in the perception of risk at a strategic level, previously defined as internal risk. The accident is generally linked to the coexistence of several events, whose occurrence is always highly likely.

3.1.3.5 The compensatory mechanism

The aforementioned topics lead to the hypothesis that, in a potentially risky situation, human behaviour is linked to a "subjective balance", which the person adjusts from time to time, in accordance to the variation of the perceived risk. This can occur for two reasons:

1) a sudden change in the boundary conditions that is perceived in the short term and is not characterized by stability;

2) a structural change in the risk itself, which tends to persist with a certain stability and to influence the subject in the medium or long-term, affecting his attitude while driving.

As mentioned in the previous paragraph, in this theory, the R risk can be considered as the sum of two components, Re and Ri[79]:

$$R = Re + Ri \qquad \text{(Eq. 3-3)}$$

At this point, it is possible to analyze individually both the external risk (Re) and the internal risk (Ri).

The external risk could be defined as a "measured" or "measurable" risk, a "lived" risk, i.e. a risk which presents itself at time t = $\underline{t}$ and determines an immediate and effective response by the user: a decrease in speed.

For example, at an intersection, this risk depends both on the actual risk and the perceived external risk. The actual risk is linked to its presence, the perceived external risk is linked to the road characteristics (distance at which it becomes visible, vertical warning signs, lighting, approaching road markings, etc.). For example, in case of a failure of the street lighting system, the user will react with a sudden speed reduction. Therefore, the external component generates a cause-effect relationship between action and result (if there is a sudden lack of visibility, the user immediately slows down because he considers a certain likelihood of an accident).

Re corresponds to the balance between the driver's perception of the drive-through reality and the actual external environment. In general, it can be stated that:

$$Re = Rre - Rp \qquad \text{(Eq. 3-4)}$$

Where Rre is the Real External Risk and Rp is the Perceived Risk. When the balance is realized, the perception corresponds to reality, and the external risk is zero.

Therefore, after unexpected changes in boundary conditions, also the real external risk (Rre) varies, becoming very different from the perceived risk (Rp). Critical situations occur when Rre increases in a very short time.

The internal risk, on the other hand, is the result of a comparison between the Safety budget (bS) that a driver is willing to spend for the journey, and the perceived internal risk achieved, over time, by mental induction. It should be noted that, in this case, the previously defined target level of risk (Wilde's theory) is labelled as "Safety budget" to introduce the target risk in a compensatory framework with other risks faced by the drivers, with a "monetary" meaning of the original acceptation of target risk. The component of perceived internal risk is elaborated through heuristics, which means that it is an outcome deriving from experience. The relationship between action and effect is not certain, due to the lack of knowledge of some information: it is possible that a user may decide not to slow down in lack of lighting because he/she knows the path or because he has similar previous experiences. Therefore, even if he/she does not recognize the danger, he/she does not run into problems.

Ri corresponds to the balance between a driver's perception and his unconscious reality. In general:

$$Ri = Rp - bS \qquad \text{(Eq. 3-5)}$$

Rp is the perceived risk and bS is the Safety Budget.

A user decides his/her "average driving attitude" according to the conditions around him/her. In this case, the user evaluates Ri, as in all other situations in human life, and this depends on the condition of the driver and

[79] A detailed description of External Risk and Internal Risk is given in Chapter 4.

his/her aversion to risk. If the user drives safely, Ri must be null or negative if he/she is particularly risk-averse:

$$Ri \leq 0 \rightarrow Ri = -\Delta i \qquad \text{(Eq. 3-6)}$$

Δi is a generally a positive term and it expresses the risk aversion of the subject.

Based on the above considerations, replacing (2) and (3) in (1), the new equation is expressed as follows:

$$R = Re + Ri = Rre - Rp + Rp - bS \qquad \text{(Eq. 3-7)}$$

The total risk depends on the External Real Risk and on the Risk Budget and not on the perception of risk. This would lead to the conclusion that, in order to reduce the risks, there is the need of acting only on factors that could give rise to sudden risks (road geometry, road signs, vehicle maintenance, driver's health), or on factors that could influence the personality of the driver.

Re and Ri act independently on human behaviour: Re acts on immediate reactions in a very short time, while Ri influences unconscious reality. Therefore, these risks influence two different processes and cannot be considered equal. The term Rp of (Eq. 3-4) is different from that of (Eq. 3-5), so that the two terms cannot be null in (Eq. 3-7).

The internal perception of risk acts at the level of strategic behaviour and affects the user's behaviour at a specific point, the rest of the route and partly the subsequent routes too.

Engineering measures can therefore influence the external risk at the level of effectiveness and the internal risk at the level of experience (direct, indirect or emotional). It may seem that the real risks can be mitigated by positively acting on the perceived internal risk: flattening a curve or highlighting an intersection. As a result, a general increase in speed may occur with the same bS. This increase is further encouraged if the direct experience is also positive (a route familiar driver frequently experiences a specific driving behaviour with a positive result).

However, some situations may be neglected by unconscious processes. For instance, the behaviour of imprudent or aggressive users is often not properly evaluated because it is considered rare.

The long-term compensation is a result of how the negative event is "felt" on the basis of one's own experiential learning, that influences the perception of risk.

The internal perceived risk is affected also by the possible asymmetry of judgment caused by experiential learning.

The analyses described are shown in Table 3.4.

Tab. 3.4: Elements influencing the different risk components.

		Probability	*Damage*
External real risk	Rre	Characteristics of a road section	Living costs related to the accident, estimated cost of a human life
External perceived risk	Rpe	Features of visibility, signage, lighting, user-specific driving training	Living costs related to the accident, estimated cost of a human life
Internal perceived risk	Rpi	Direct and indirect experience, emotional reaction, personal characteristics	Perceived cost of the accident (economic cost and cost of living)
Safety budget	bS	Motivation for travel, economic capacity, economic cost of travel, gender, age, risk appetite	Perceived cost of the accident (economic cost and cost of living)

3.1.3.6 The economical approach to the compensation process

The outcomes of the aforementioned decisional program are behavioural components. After the risk estimation and the comparison with the target risk (or Safety budget, as previously defined), the user would choose his/her own speed and trajectory according to the own risk aversion.

The mechanism that allows to switch from the risk perception and estimation to the specific behaviour is still undetermined. This switch could be achieved through an economical approach, as Peltzman (1976)[80] and Tarko (2007)[26] have proposed.

A strong support to the theories about the perceived risk as the metric for the driving behaviour, could be of course constituted by clues from on-board vehicles safety devices.

One of the first studies in this field was the one run by Peltzman (1976)[80], made for the evaluation of the effects of the new U.S. regulation in 1960, about the compulsory safety devices installed on vehicles. The research mainly demonstrated that the road death rate has not affected at all by the regulation addresses: the death rate was indipendent by the countermmeasures written in the regulation. This was in contrast with the regulation goal. It was blatant, on the other side, that there was a switch in crash occurrence from crashes involving only drivers to crashes involving pedestrians too, with a crucial increase in damage severity. From this analysis, Peltzman (1976)[80] elaborated the economic model of the risk compensation, where the user behaviour is considered as fully rationale. At the bottom of the model there are some principles of the "decision making" theory, based on the balance between the "crashes likelihood" and "driving intensity" (speed, emotion, carelessness, etc.).

The measures of on-board passive safety act on this balance, as it is shown in the graph, Figure 3.9.

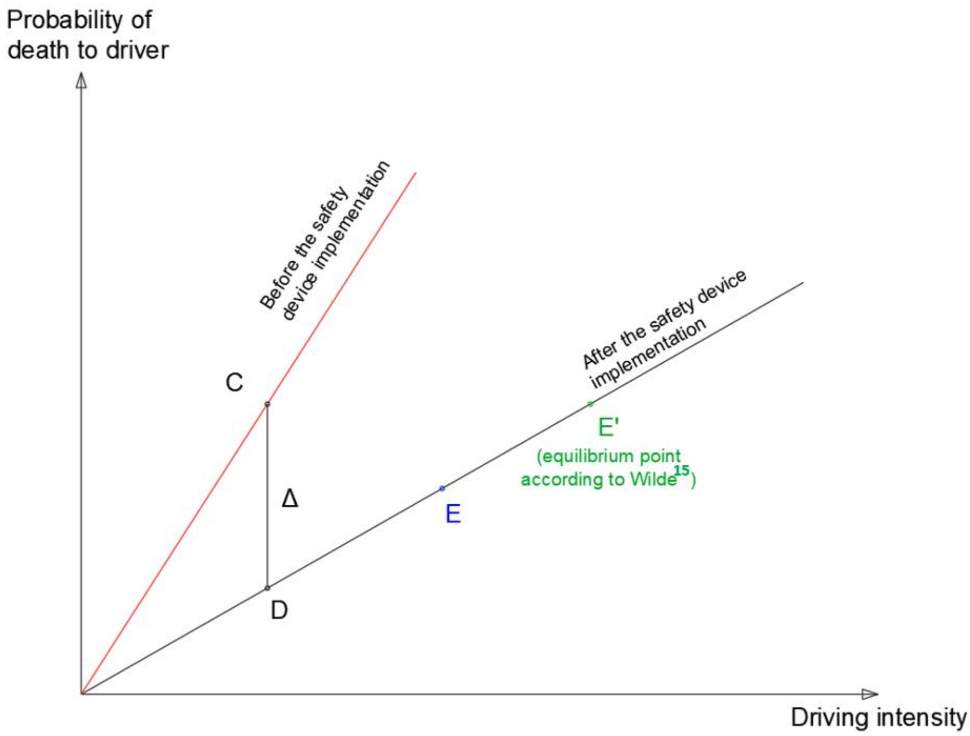

Fig. 3.9: Likelihood of perceived death vs Driving intensity Diagram (based on Peltzman, 1976[80]).

The "death likelihood" and "driving intensity" perceived by the user, has a trend affected by the user's utility goal. When safety devices were installed at vehicles, the death risk, which affects the driving intensity, decreased. The "benefit" of the measures is represented by the difference between C and D. However, Peltzman (1976)[80] assessed that this decrease would correspond to a compensational behaviour by the driver. This process consists in driver needs of increasing the driving intensity to compensate partially or fully the acquired "benefit". So the new equilibrium, "consumption equilibrium would be in point E rather than in D (Peltzman, 1976[80]). The cost variation of the driving intensity (obtained after the decrease of the death likelihood if crashes occur), would lead to an increase of "consumptions" and so of risks for safety. Peltzman's theory does not express the rate or the criteria of this compensation, but he finds out on one side, the relationship between *death likelihood* and *driving behaviour* (providing a qualitative display on the indirect measurement of the perceived risk). On the other side, he attempted to explain the utility concept and how it takes action in the compensation phenomenon.

[80] Peltzman S. (1976), "Toward a More General Theory of Regulation", *Journal of Law and Economics*, 19(2), 211-240.

The same considerations were made by Tarko (2009)[17]. He neglected the explanations about the inner motivations of the compensation process, focusing on how the equilibrium intoduced by the compensation could appeat in actual driving behaviour, particularly in the speed choice. Tarko (2009)[17] defines the disutility concept related to the perceived travel costs: the subject always acts seeking for minimizing the expected cost for his/her activities. Alternatively, it could be said that the user would change his/her behaviour in front of driving environment changes in order to maximize his/her own utility.

In few words: why does the user decrease his/her speed while increasing the perceived risk? How does the speed induce uneasiness in the user?

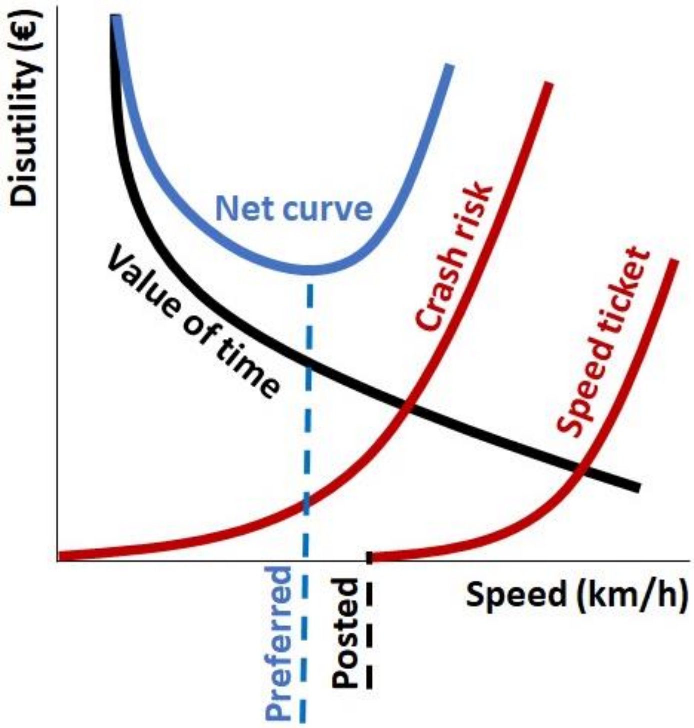

Fig. 3.10: Disutility vs Speed Diagram (based on Tarko, 2007[26]).

According to Tarko (2007)[26] the travel disutility is equal to the sum of three components:

1. The subjective cost of the travel time
2. The travel perceived risk with its potential consequences
3. The severity perceived by the speed limit checks.

The sum of these three components is shown in Figure 3.10.

The descending branch of the curve overlaps with the idea that the user "feels" that driving at very low speeds is frustrating and a waste of time. The main reason of speeding up is minimizing the time taken to get to the destination where there are activities to be done. While speeding up, the perceived risk of crashing also increases, though. It increases as fast as the speed increases. In this way, Tarko (2007)[26] tried to explain how the phenomenon of adjusting speed would be defined by a mechanism of risk adjustment. The perception of the user about the checks of the imposed speed limit makes the driver slow down. Also, Wilde (1994)[15] uses the economics to estimate the perceived risk and to show his theory about the compensative process. As previously aforementioned, the homeostatic risk theory starts from the assumption that in each activity, people accept a risk level subjectively determined in relation to own safety, health – or whatever they care for. This accepted risk level is in exchange of the benefits they hope to receive by the activity they are doing (moving, eating, working, walking, studying,..).

People alter their behaviour in response to the implementation of health and safety measures, but the riskiness of the way they behave will not change, unless those measures are capable of motivating people to alter the amount of risk they are willing to incur (=target level of risk). (Wilde, 1994[15])

The Wilde's theory relies on a compensative criterion as described by Peltzman (1976)[80], even if in this case the compensative mechanism is partially unconscious and not necessarily rationale.

Referring to the Figure 3.11, Wilde (1994)[15] predicted that the new equilibrium point should be in a point, E', i.e. where the risk level chosen, in case of safety device-implementation, is assumed as constant if compared with the red line. He tries to explain why the risk level is different from the null value, 0. He used the graph in Figure 3.11 to meet this purpose.

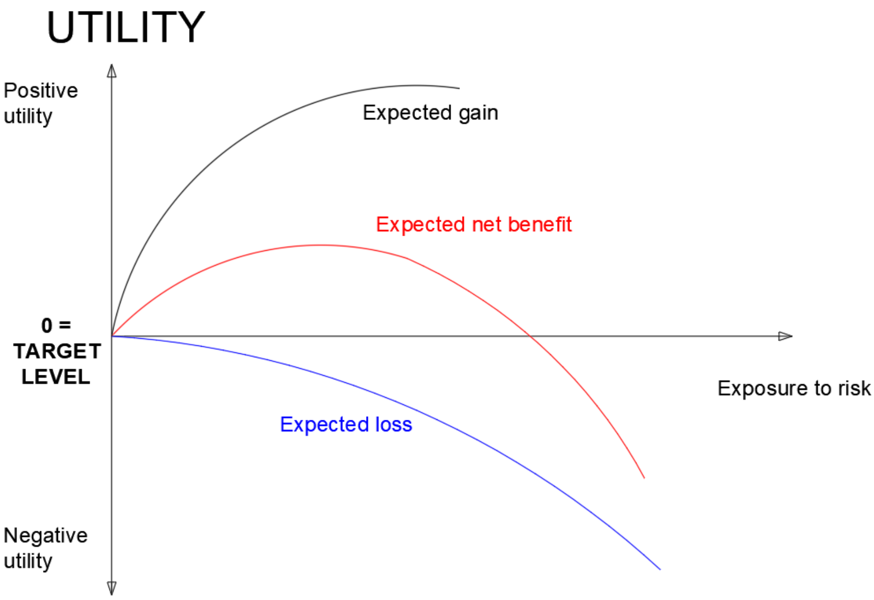

Fig. 3.11: Utility vs Exposure level Diagram (Based on Wilde, 1994[15]).

The horizontal axis stands for the increase of level of exposure to risk (due to the increase of speed or of the driving complexity). The two curves y_1 and y_2 represent respectively the benefits and the expected costs after the activity. The sum of the two curves generates the curve y_3, which represents the net effective benefit. This curve has a bell trend: it increases till a peak and then it decreases.

The expected costs exceed the potential income for extremely high risks making the benefits negative: the majority of people chooses an average position between the two null-gain thresholds. Each user tends to optimize his/her own exposure to risk, so to achieve a risk level where the user needs are satisfied, so neither the perceived risk removal nor the reduction in driving difficulties.

3.1.3.7 Infrastructural engineering role in compensation

The engineering countermeasures are significant for road safety, but their effectiveness in the global risk perception must be further analyzed. The effect of the perception about an inner risk, if it has an opposite sign to the one of an external risk, could lead to the intervention ineffectiveness.

The most blatant case is the one of lighting at intersections. Several experiments have showed that implementing lights at intersections lead to a speed increase, and thus the risk and the severity of a potential crash increase too.

Lights reduce the likelihood of the external risk, and ensure the complete perception of the intersection. But due to the heuristics effect, lights may induce comfort to the driver, who feels erroneously to be in a safe environment. The unpredictability of other users' behaviours or the user capabilities are assumed as constant in cases like this to reduce complexity.

Hence, the interventions aiming at managing road safety should be studied taking into account the proposed model.

In case of positive ΔV, interventions to mitigate and decrease compensational effects are necessary. For instance, relating to the already explained case of lighting, a mitigation strategy could be the installation of speed cameras. In fact speed cameras might increase the inner risk due to the increase of new costs linked to them.

In this regard, making evident the risk perception through the countermeasure is a very effective choice. In this case, the countermeasures would likely determine reactions, particularly whether dealing with the external risk. For instance, rumble strips on the edge of the carriageway generate noise and jolts when the user goes off the road. In this way, the user is aware of what is happening and perceives the risk perfectly. In this case, the perceived risk would overlap the real risk. Further studies could investigate road interventions and their impacts on the perceived inner risk and external risk in order to make proper and efficient evaluations. This is aimed at designing a "global" intervention matching with the huge complexity of the user behaviour.

3.2 Factors influencing crashes[81]

The frequency and severity of crashes are affected by several factors, firstly by the *exposure* to the crash risk. Of course, any human activity is exposed to the risk of accidents, so in the case of mobility, the exposure generally refers to the amount of travel, i.e. the number of kilometres travelled. However, there are various ways in which these can be travelled: as a pedestrian, as a cyclist, driving a car, taking a bus, etc. Each of them is characterized by a different level of crash risk.

The risk depends not only on the mode of transport but also on the amount of people using the same mode of transport. For example, the higher the proportion of pedestrians in traffic is, the lower the risk can be. Thus, the probability of an accident is influenced by a high number of risk factors related to the traffic elements: vehicles, infrastructure and road users. A risk factor is an element that increases the probability of a crash. However, not all factors are strictly "causes" of accidents: the statistical association is a necessary but not sufficient condition for the existence of a causal relationship.

Whereas, deaths, personal injury or property damages are indicators of crash severity.

Traffic volume represents the number of road users (pedestrians, cyclists and car users) which use a road every unit of time. A multivariate analysis of some factors that influence the monthly number of crashes with injuries in Norway revealed that variations in traffic volume are accountable for about two-thirds of variations in crashes (Elvik and Vaa, 2004[59]), followed by other minor factors (such as e.g. county and weather and daylight).

Hence, traffic volume seems the most important factor which affects the number of crashes. For the same study in Norway, the traffic volume effect on crash severity has been studied, as it is qualitatively shown below.

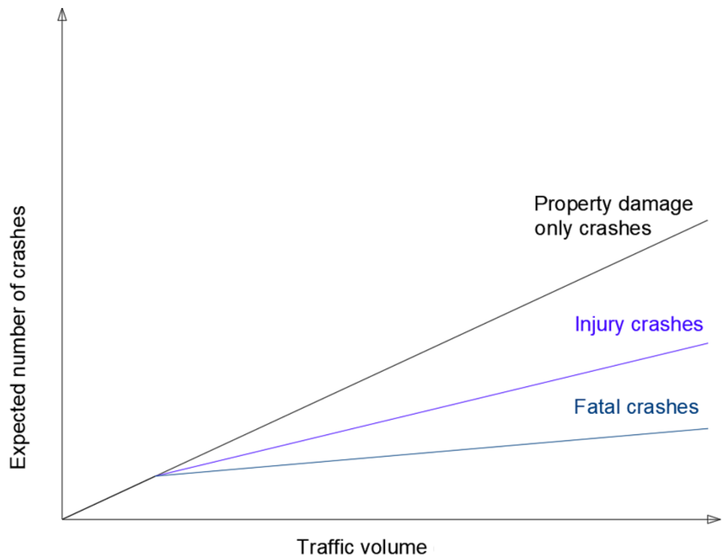

Fig. 3.12: Typical relationships between traffic volume and expected number of crashes (based on Elvik and Vaa, 2004[59]).

The term "road traffic" does not just mean "vehicles", but it considers all the categories of road users actually

[81] In this section some concepts are exposed, firstly published in the book: Elvik R., Vaa T. (2004), *The Handbook of road Safety Measures*, Elsevier, Amsterdam, Nederland, except than where elsewhere specified.

on a road. Brüde and Larsson (1993)[82] have produced an estimation of crashes thanks to the following composite exposure measurements:

Number of accidents involving road users 1 and 2 = $\alpha Q_1^b Q_2^c$ (Eq. 3-8)

Where Q_1 is the number of type 1 road users, Q_2 is the number of type 2 road users, b and c are coefficients (the first for motorvehicles: 0.50 in case of accidents involving pedestrians and 0.52 in case of accidents involving cyclists, the second for pedestrians/cyclists: 0.72 in case of accidents involving pedestrians and 0.65 in case of accidents involving cyclists); while α is a constant.

Based on this report, the number of crashes can be estimated for any combination of traffic.

3.2.1 Other risk factors statistically associated with the accident rate

According to Elvik and Vaa (2004)[59], there are several other factors which were statistically associated with the accident rate (number of accidents per unit of exposure). The relevant aspects highlighted by the same authors for each factor (as based on literature studies) are briefly summarized as follows.

For the involvement in accidents:

- **Road type or road context.** Freeways generally have the lowest crash risk compared to all other road types. Rural areas in general are affected by less crashes than urban areas, where the crash rate could be greater also of one order of magnitude than in rural areas. Whereas, collector and access roads are dangerous in both urban and rural areas.
- **Road design elements.** Number of lanes, road width, vertical and horizontal curves, design of intersections and other punctual elements are influential design elements, with different impacts on crash frequency. For instance (Figure 3.13), narrow curves are dangerous and increase the crash frequency. Their effect may change according to the main characteristic of the horizontal alignment, though: it is lower in a winding road than in a straight road, because it can be expected and the speed is already low.
- **Environmental risk factors.** The increased risk of accidents is also determined by: darkness, rain, adverse road surface conditions, as assessed by numerous studies. Dark conditions have a great impact on crash occurrence, especially for pedestrians. Instead, from dry road to snow-covered road the crash rate significantly increases.
- **Age and gender of the driver.** Young drivers and men tend to have a higher accident rate than women. In the adult age, the male crash rate seems to fall below the female rate. This result could be explained if women drive much less than men, drive smaller cars that provide less protection and on urban roads where the crash risk is higher.
- **Health conditions of road users.** The crash rate generated by various health problems and illnesses is generally very low, except for the use of drugs, senile dementia and a reduction of the useful field of view. The fact that other illnesses have a low impact on the crash rate could be caused by drivers trying to compensate by adopting more cautious behaviours.
- **Alcohol and drug use.** Using alcohol and drugs strongly and dramatically raises the crash rate. This increase is remarkable for fatal/injury crashes, more than for property damage only crashes.

For the accident severity:

- **Type - mass of the vehicle.** A heavy vehicle would provide drivers with greater safety against injury or fatal crashes. An official Norwegian crash statistic confirmed this, highlighting how the probability of getting injured after a crash is higher for vulnerable users (see Fig. 3.14). However, a heavy vehicle generates more protection for the driver, but more danger for other users with smaller vehicles.
- **Impact speed.** The severe injuries likelihood in a crash depends on the speed during the impact; obviously not using the seatbelt increases the crash severity. At high speeds, severe crashes are not avoidable without

[82] Brüde U., Larsson J. (1993), "Models for predicting accidents at junctions where pedestrians and cyclists are involved. How well do they fit?", *Accident Analysis & Prevention*, 25(5), 499-509.

seatbelts. Pedestrians and bikers are exposed to crash severity in the same way, but severe injuries happen at significantly lower speeds.

- **Use of protection systems.** The use of protection systems is very important for road safety, especially for unprotected users such as pedestrians, cyclists, motorcyclists. Using the helmet or protective suits on motorcycles reduces the injury likelihood (especially if used together). Comparable benefits are for car users (also back-seat passengers) using seatbelts.

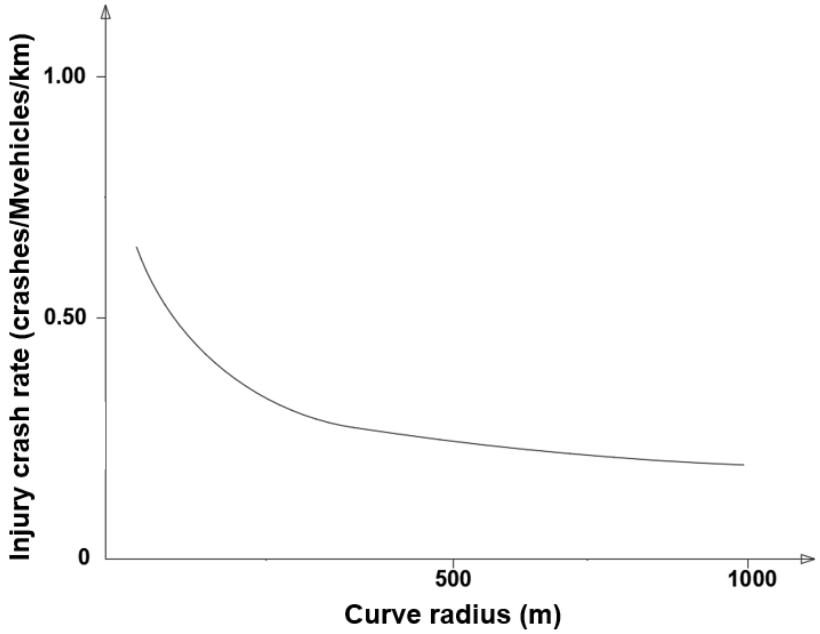

Fig. 3.13: Relationship between radius of curvature and injury crash rate (based on Elvik and Vaa, 2004[59]).

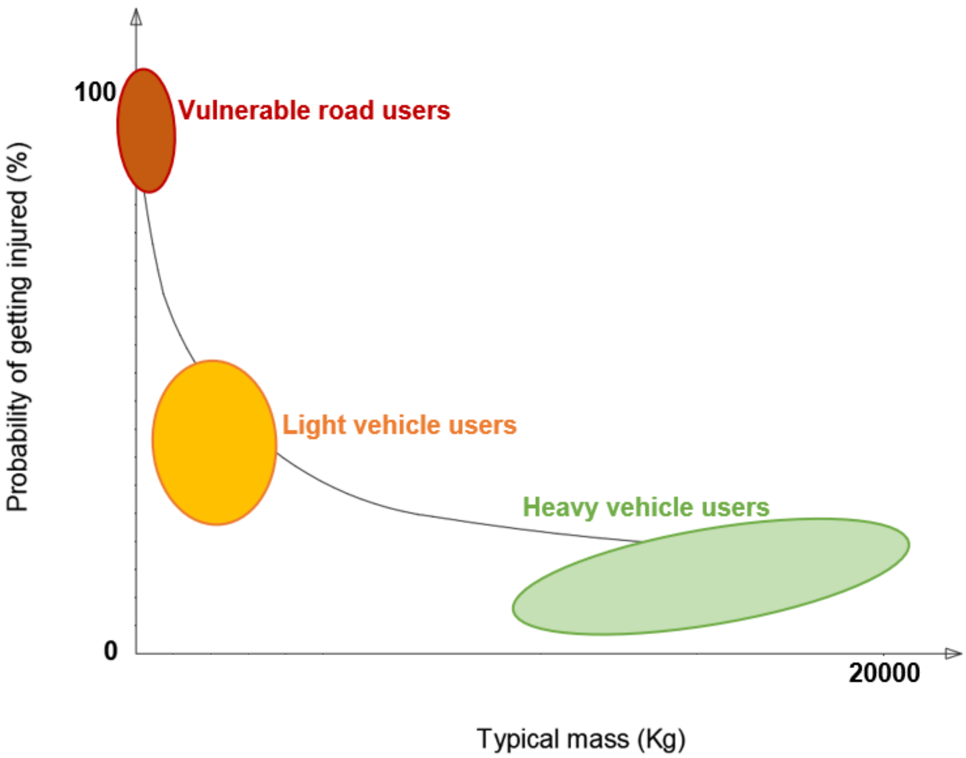

Fig. 3.14: Relationship between accident probability and vehicle type (based on Elvik and Vaa, 2004[59]).

To define the importance of each risk factor, the "*Attributable Risk*" can be defined. It is the fraction (assuming values within a range from 0 to 1) of crashes or injuries that a given risk factor causes. Alternatively, this fraction can be interpreted as a measure of the reduction in the number of accidents or injuries that can be achieved by removing that factor. For instance, unprotected users may be 10 times more likely to be at risk than protected users, according to the attributable risk of a fatal accident for unprotected users.

In the following paragraph, a specific focus on the influence of the impact speed factor is provided, given its well-known importance.

3.2.1.1 Focus on impact speed

Relationship between speed and road accidents: The Power Model

What are the consequences of changing the average speed on the accident frequency and severity?

The "Power Model"[83] is useful to answer to this question. It is characterized by six equations which relate the average speed of vehicles (S) to the number of accidents (A) or to the number of casualties (C)[84]:

- Number of fatal accidents $$A_1 = \left(\frac{S_1}{S_0}\right)^4 A_0 \quad \text{(Eq. 3-9)}$$
- Number of deaths $$C_1 = \left(\frac{S_1}{S_0}\right)^4 A_0 + \left(\frac{S_1}{S_0}\right)^8 (C_0 - A_0) \quad \text{(Eq. 3-10)}$$
- Number of fatal and serious injury accidents $$A_1 = \left(\frac{S_1}{S_0}\right)^3 A_0 \quad \text{(Eq.3-11)}$$
- Number of fatalities or serious injuries $$C_1 = \left(\frac{S_1}{S_0}\right)^3 A_0 + \left(\frac{S_1}{S_0}\right)^6 (C_0 - A_0) \quad \text{(Eq. 3-12)}$$
- Number of injury accidents (all) $$A_1 = \left(\frac{S_1}{S_0}\right)^2 A_0 \quad \text{(Eq. 3-13)}$$
- Number of injured road users (all) $$C_1 = \left(\frac{S_1}{S_0}\right)^2 Y_0 + \left(\frac{S_1}{S_0}\right)^4 (C_0 - A_0) \quad \text{(Eq. 3-14)}$$

Updated models

Even if the power model looked like to be a reliable model, according to a study conducted in 2009[85], the most recent assessment of the model (2013[86]) supported the idea that the Power Model is incomplete. This finding is justified by the absence of the initial speed influence on the crash rate in the original model, while the exponent should be significantly affected by the initial speed besides of speed changes. In the latter case, there is a significant change in crash rates whether the same speed variation happens with a very high or a very low initial speed.

According to what has been said, a re-parametrization of the model was fundamental, in which exponential functions were fitted to data points (changes in speeds and accidents) considering different initial speeds. By the way, no clear relationship were detected between initial speeds and exponents, in a first moment[85].

[83] Nilsson G. (2004), *Traffic safety dimensions and the power model to describe the effect of speed on safety* (Doctoral dissertation, Univ.).

[84] Index "0" for the values observed before the change of the average speed, index "1" for the ones observed with a new average speed.

[85] Elvik R. (2009), *The Power Model of the relationship between speed and road safety. Update and new estimate, Report 1034*. Institute of Transport Economics, Oslo.

[86] Elvik R. (2013), "A re-parameterisation of the Power Model of the relationship between the speed of traffic and the number of accidents and accident victims", *Accident Analysis & Prevention*, 50, 854-860.

 ISBN 9788835111863

Previous studies[87, 88] have shown that the exponential model could fit better than the Power Model the data.The exponential function has the following baseline equation:

$$E(x) = \alpha \, e^{\beta X} \quad \text{(Eq. 3-15)}$$

Where α and β are the coefficients estimated by means of regression analysis taking into account the initial speed and the speed variation. The fitting was calculated both on weighted and not-weighted data. The weight was computed accordingly to speed intervals (12 intervals with 10 km/h difference). The results were found to be robust for both the cases (weighted and not-weighted data). The estimated parameters (weighted data) for Fatal crashes, Injury crashes and PDO (Property Damage Only) crashes are reported below.

Tab. 3.5: Comparisons between the Power Model results and the Exponential Model results (based on Elvik, 2013[86]).

Crash severity	*α parameter*		*β parameter*		*R-squared*	
	Power model	*Exponential*	*Power model*	*Exponential*	*Power model*	*Exponential*
Fatal	<0.001	0.072	4.234	0.069	0.987	0.981
Injury	0.003	1.983	2.124	0.034	0.986	0.994
PDO	0.010	2.928	1.911	0.032	0.989	0.992

From the Table 3.5, the Power Model fits better data than the exponential model only for fatal crashes. For all the other cases, the exponential model is slightly better than the Power Model. The two models are different especially at high speeds. The exponential models imply a greater effect of changes in speed when initial speeds are high, compared to the Power Model.

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4. Road safety and risk

4.1 Driving and attention

When there is a stimulus with the characteristics of a novelty, a reaction occurs, which modifies the subject's behaviour. On the contrary, if the stimulus occurs repeatedly, the reaction disappears and the personal behaviour may assume the "habitude" condition, without any effort, because the driver has become accustomed to it.

Recent experiments seem to confirm that the behaviour of a user driving a vehicle corresponds to habitude (driving without awareness[1],[2]) in the ordinary and everyday situation (which is, moreover, the one in which the least amount of energy is probably consumed) and it is generally without specific reactions.

This behaviour changes, leading to an attention condition whether the user has the feeling (or even the certainty) that a new element is about to appear. This new element would require a specific manoeuvre or in any case a modification of the way of driving. A typical example could be a sudden illness or a warning light on the dashboard, or the approaching phase at an intersection or the beginning of a rain event.

Basically, the behaviour of a user driving a vehicle may depend on habit, attention or reaction.

The occurrence of a road crash is due to at least one unexpected event. Indeed, obviously if the event was predictable, it would be avoided, and the negative consequences deleted. So, what does the unexpected event depend on?

Under normal conditions, the user drives along the road in a vehicle, in a given environment, with known travel conditions. His/her driving behaviour thus leads back to a standard behaviour which takes into account presumed repetitive conditions for each involved factor. In other words, he/she is driving with a behaviour appropriate to a user with a good psycho-physical level, in an efficient vehicle, along a well-designed road, under regular traffic conditions and in a good environment.

When this ideal situation does not occur, due to a long-lasting condition (for example the user has a headache, or the vehicle does not accelerate or the road is not surfaced or there is steady traffic or persistent rain), at the beginning there is a reaction which modifies the user behaviour quite easily, and this reaction then becomes normal (for example reduction in speed or greater attention while overtaking, etc.).

On the other hand, the reaction certainly occurs when a new element suddenly and unexpectedly appears in any of the involved factors, causing the immediate removal of the accustomed conditions and habitual behaviour, without going through the intermediate attention phase.

The new element should be related to a finite difference between the user's expectations (which would cause habitude) and reality, which requires a different behaviour.

The individual capacity of adapting to this difference may be more or less rapid. Thus, modifying the driving behaviour might either be sufficient or insufficient to avoid any negative consequences, deriving from the presence of the unexpected element. If there is no time for the manoeuvre or the manoeuvre itself is not successful, the crash will occur.

The phenomenon may be more severe if more than one unexpected element happens at the same time, because the simultaneous presence of two distinct reactions is much more unlikely manageable (because the stimuli may contrast one with each other). It requires more time for analysing the situation, more problems when taking a decision so the reaction and the manoeuvre would be slower and more uncertain.

[1] Charlton S. G., Starkey N. J. (2011), "Driving without awareness: The effects of practice and automaticity on attention and driving", *Transportation research part F: traffic psychology and behaviour,* 14(6), 456-471.

[2] Charlton S. G., Starkey N. J. (2018), "Memory for everyday driving", *Transportation research part F: traffic psychology and behaviour*, 57, 129-138.

In any case, it is obvious that the risk level will increase proportionally with differences "ΔQ" between expectations and reality, or between expected and real levels of quality and efficiency[3].

The process operated by the driver will thus be correct if the following three phases are fully completed in accordance with the following simplified order:

1) the driving behaviour of the user is usually characterised by habitude.
2) the behaviour moves to an attention stage which is sufficient to cope with an imminent change in the conditions interactively influencing the driving.
3) the behaviour coincides with a reaction to the modification, through a manoeuvre which ensures continuing to drive under optimal conditions.

However, the described process is affected by lack of attention, i.e. if the phase 2) is insufficient or absent, when an immediate reaction or a too quick one is required, starting from a habituation condition. An example could be when a driver is travelling on a wide road, with few curves, for a long time, so his/her attention decreases due to habituation. If the road geometry immediately changes because of a sequence of narrow curves, the driver will not be ready to face this new condition if his attention is not high. In fact, the driver could have missed the traffic sign requirements due to distraction. If the driver pays attention, he will read the traffic signs accurately and will slow down avoiding risky situations.

Basically, the ΔQ occurring between the phases 1) and 3) tends to have a maximum value in the absence of the phase 2), while it tends towards the minimum value (asymptotically to zero) in the presence of a phase of appropriate attention.

When phase 2) is completed, the outcome is the following equation[4]:

$$\Delta Q_{(1-2)} \cong Q_{(2-3)} \cong 0 \qquad \text{(Eq. 4-1)}$$

and thus:

$$\Delta Q_{(1-3)} = \Delta Q_{(1-2)} + \Delta Q_{(2-3)} \cong 0 \qquad \text{(Eq. 4-2)}$$

while in absence of the phase 2):

$$\Delta Q_{(1-3)} \gg 0 \qquad \text{(Eq. 4-3)}$$

If the step 2) is partially present, the ΔQ value will obviously be an intermediate value compared to the one obtained in the presence of the phase 2).

According to what has been stated above, it is evident the importance of the attention phase. The attention phase can be adequate only if the management and the control of the elements causing modifications in driving behaviour are appropriately considered.

How may the attention phase be incentivized?

- If the road is a self- explaining road;
- If there are well-designed traffic road signs;
- If the user already knows the road (familiar user).

4.2 Risk and safety cost

4.2.1 The risk

The Risk "R" is defined as:

$$R\ (€) = p \times I\ (€) \qquad \text{(Eq. 4-4)}$$

Where:

- *p* is the probability that the negative event happens,

[3] Colonna P. (2002), "Proposal for a safety function for evaluating the road efficiency level", *The third International Conference on traffic and transportation,* Guilin, China.

[4] Colonna P. and Pascazio R. (2011), "Road safety and user behaviour", *Psicotecnica, ieri! oggi? domani??* Conference Proceedings, Bari, Italy.

- *I* is the intensity of the consequences that the event would cause.

The unit of measurement of (Eq. 4-4) is money, or better, the cost of the damage caused by the crash multiplied by its probability. This equation is adopted for each kind of risk, not only the one related to road crashes. For example, if there is some medical epidemy, the risk perceived by society is related to the costs in terms of social assistance and fatalities that the society will face. Hence, the intensity of the damages due to the epidemy multiplied by their probability of occurrence give the risk (for example, considering the Covid-19, the perceived risk was high only when the probability of contagion dramatically increased). Coming back to road safety, each time a user drives his car for one km of the established route, it could be assumed that he already knows that he/she is willing to spend a quantity of money equal to p*I for that route[5]. Of course, there is a fervent hope for each user that the crash will not happen, but sooner or later it will likely happen. In case of crashes, all the money saved during the previous journeys in which the crash did not occur, will be spent all at once.

Therefore, talking about risk R is equivalent to dealing with the money that a person is willing to spend to avoid or to reduce the risk (i.e. for safety). This quantity of money is called in this section "Safety Budget". This concept is similar to the "target level of risk" identified by Wilde (1982)[6], which is the target risk specific to each driver and it is constant in the long-term period. However, in this case it is called "Safety Budget" to point out that it could be actually quantified and related with other similarly computed monetary measures, besides of being flexible over time.

While travelling, behavioural patterns are planned (more or less risky for safety) based on how much drivers are willing to spend for that specific trip. Since each travelled kilometre entails a travel cost "cu" for the user, it is also possible to calculate - for each behaviour and therefore for each speed v - the total cost Cu(v) for the user, sum of the monetary travel cost cu(v) and the risk-related costs:

$$Cu(v) = cu(v) + p(v) \times Iu(v) \qquad \text{(Eq. 4-5)}$$

where p(v) and Iu(v) represent – per Km – the crash likelihood at the speed v and the intensity of consequences for the user, in case of a crash occurred while driving at the speed v.

Actually, Cu(v) consists of several addends:

- cost c_1 related to the vehicle (fuel, amortization, insurance, maintenance, etc.)
- cost c_2 related to the user (value of used time)
- cost c_3 related to the road (toll)
- cost c_4 related to any sanction for traffic violations
- cost c_5 related to a possible crash.

It is necessary to take into account that c_1, c_2 and c_3 are spread over time, while c_4 and c_5 are concentrated.

Obviously, the same relation can be found considering the costs for the whole society and not only those for the single user, that is the monetary travel cost cs(v) and the total cost Cs(v). In this case, it is necessary to take into account a cost c_6, related to the environmental impact of mobility, and a cost c_7, related to the road crashes prevention and sanctions for crashes themselves. It would be:

$$Cs(v) = cs(v) + pa(v) \times Ia(v) + ps(v) \times Is(v) \qquad \text{(Eq. 4-6)}$$

Where:

- pa(v) and Ia(v) are the crash likelihood (per km) and its consequences for the society, at the speed v.
- ps(v) and Is(v) are the sanction likelihood and its consequences for society, at the speed v.

Users who give importance only to their own utility and who are sure of not suffering a crash would travel at a speed v, minimizing cu(v). On the contrary, users who take into account the possibility of being involved in a crash would choose v, minimizing Cu(v).

At the end, "social" users – taking into account the needs of the whole society – would choose v, minimizing Cs(v)[7].

The user behaviour (related to the km to be travelled) naturally takes into account the boundary conditions in order to ensure that the risk stays constant. For example, if the user psychological and physical conditions are not

[5] Bureau of Infrastructure, Transport and Regional Economics, BITRE (2010), *Effectiveness of measure to reduce road fatality rates, Information sheet 39*, BITRE, Canberra.

[6] Wilde G. J. S. (1982), "The theory of risk homeostasis: implications for safety and health", *Risk analysis*, 2(4), 209-225.

[7] In (Eq. 4-5), we assumed c_1, c_2, c_3 incorporated in *Cu(v)*, but clearly there are other possible interpretations. For example, they (or a portion of them) may be part of *Cs*.

good, since this scenario would increase the probability of crashes, the user is forced to slow down, restoring the crash likelihood at the initial accepted value.

The same mechanism could be triggered by the vehicle (e.g. if tyres are worn), the environment (e.g. if the weather gets worse), the traffic (e.g. if it increases). The speed reduction which is required by the inner behaviour to maintain a constant risk level, inevitably leads to an increase in travel time and therefore to higher travel costs. In this way, it is like deciding to spend money to reduce the crash risk and the cost of this reduction can also be theoretically quantifiable. Reducing the travel speed means taking the decision to pay more for the ticket of the journey itself and to spend less money on charges and insurance against the crash risk and potential sanctions. Increasing the travel speed means deciding to pay less for the ticket for the journey itself, but to spend more money on charges and insurance against the crash risk and potential sanctions.

However, the driver psycho-physical conditions, the vehicle conditions, weather and traffic conditions are generally well-predictable in advance, with some exceptions. If the driver is not a regular user, the road conditions are evaluated at the moment of driving. In any case the user is aware of the existence of uncertainties about the conditions he will effectively meet on the route.

Therefore, the uncertainty is transformed into less risky behaviour (e.g. with a slightly lower speed), rather than the risky behaviour that would have been adopted perfectly knowing all the boundary conditions a-priori. This means that the monetary risk that the driver actually spends on the route will be less than the risk corresponding to the perfect knowledge of the boundary conditions.

The difference between the two risk values represents a δ of money which is not spent. On the contrary, this is held to be spent in unforeseen scenarios that will certainly occur because of the above described uncertainties.

The curve below[8] represents the disutility due to the time wasted while travelling against speed.

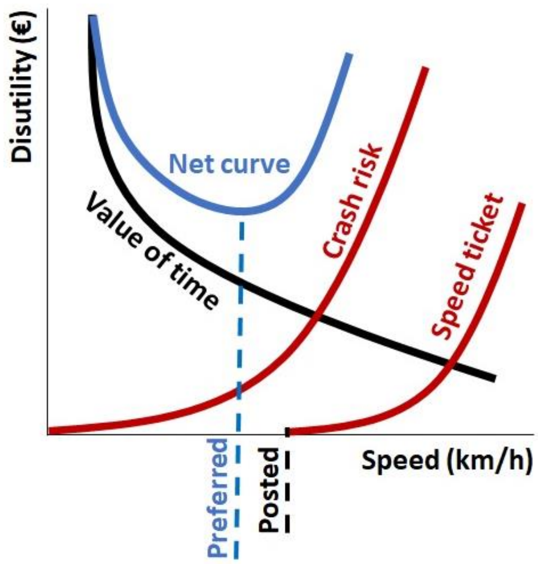

Fig. 4.1: Synthetic relationships between perceived disutilities and speed selection (based on Tarko, 2007[8]).

According to Tarko (2007)[8], the total travel disutility is the sum of three components, the subjective costs of travel time, the perceived risk of accident and its potential consequences, and the perception of speed limit enforcement:

trip disutility = time value + perceived risk + perceived enforcement

Actually, the value of time is not only subjective (it depends on income), but it is a function of the user conditions (e.g. the value of time on Sunday or during a holiday is different from the value that the same subjects give to their time on a workday). As a consequence, the curve can be very flat during non-working periods. In these cases, the speed selection is dominated by other factors rather than by the time wasted. The most significant factors should be found in the main motivations for a trip. For example, in the case of a tourist trip, low speeds can be determined by the pleasure of enjoying the day talking with the passengers, enjoying the landscape with

[8] Tarko A. P. (2007), "Estimating Subjective Risk Revealed Through Driver Speed Choice", in a talk given to the "Road Safety and Simulation International Conference RSS 7, 8, 9th November 2007", Rome, Italy.

a greater intensity, etc. Instead, if the aim of the travel is to bring someone to the hospital, as soon as possible, the time value will be really high, and the speed would increase drastically to save time. Therefore, a function of variability for the value of time should be sought, in order to take into account all these factors. The same is true for the curve relative to sanctions and probably also for the risk-related curve.

However, the use of Eq. 4-4, in order to analyse the risk R with the objective of lowering its value, can be misleading. In fact, focusing the attention on the probability and consequences of an event means to evaluate how many times in a certain time and with what intensity the event occurred, regardless of how and why the event occurred.

This can lead to underestimating the importance that the human behaviour has in relation to the event, while it is clear that the road crash is strongly influenced by the user's decisions.

4.3 Safety budget and perceived risk

Many models have been hypothesized to be able to represent in a schematic way the road user behaviours related to the crash risk.

One of these best-known models is the Homeostatic Theory of Risk (HRT) by Wilde G.J.S. (1982)[6]. This theory was already mentioned in the previous chapter 3. However, it is crucial for the presentation made in this chapter and then, its main characteristics are here summarized again. The HRT model by Wilde states that the user, time by time, makes a comparison between the Perceived Risk Rp and his Safety Budget bS (as previously explained, this quantity is called "target level of risk" by the author), thereby modifying his/her own conduct according to it.

For example, if the user perceives that the crash risk, due to the presence of a curve, may increase up to exceeding his/her accepted risk level, he/she will be forced to decrease the perceived risk by reducing the speed, thus bringing back the value of Rp exactly equal to the Safety Budget bS chosen by him/herself (Figure 3.7 in chapter 3).

This process takes place moment by moment, but it is affected by the bS value which is actually a benchmark and therefore a constant for that specific user. It is a constant not only in that specific moment but also in a sufficiently long-time interval close to the moment considered, at least until the psychophysical conditions of the user are unmodified.

Wilde himself admits that changes in the boundary conditions cause variations in the user accepted Safety Budget, but only after a certain time, useful for the driver to verify the stability of the new conditions.

It is therefore evident that Wilde's Homeostatic Risk Theory (HRT) is applicable when the accepted Safety Budget becomes stable in the subject and this may have happened after a long time (for example if keeping the speed within the limits and following the road rules to avoid fees become a habit) or after an medium-term interval (e.g., when the subject, being forced to drive despite of a headache, decreases his accepted level of risk in the new and worse health conditions).

The Safety Budget is therefore an internal constant of the subject, and, according to the subject him/herself, it is the benchmark for the Perceived Risk measured moment by moment.

This homeostatic equilibrium should ensure a safe behaviour, at least in the expectation of the subject.

It should be recalled that the "Homeostasis" is the equilibrium of living systems which are affected by continuous changes.

4.4 External risk and internal risk

Despite of what has been aforementioned, crashes do happen.

Why?

The answer could be because the Safety Budget stands for the consequences that the user accepts to handle after the occurrence of the crash caused by him/herself. It is so evident that the overestimation of the consequences is unlikely and rare. Exceptions are represented by crashes caused deliberately (suicide) or by emergency travels (to the hospital) Some crashes happen due to sudden events, in which the equilibrium conditions are quickly or very quickly lost. On the other hand, it is obvious that, apart from crashes caused deliberately (e.g. suicide) or totally unforeseeable, with no detectable risk (e.g. a billboard falling on a moving vehicle), all other crashes happen because of an underestimation of the perceived risk linked to an event. In fact, if the driver had been able to perceive and predict what could have happened, he certainly would have done everything to avoid the crash. In these cases, obviously, the problem is not in the Safety Budget, but in the Perceived Risk, which is underestimated compared to the External Real Risk.

The previous remarks lead to considering the hypothesis that, in a potentially risky event, human behaviour is linked to a "subjective equilibrium". This equilibrium is updated by the subject if the Perceived Risk changes. This may happen for two reasons: a sudden change in boundary conditions, perceived in the short term and not characterized by steadiness, or a structural change of the risk itself, which tends to persist with a certain stability and influences the subject in the medium or long term, affecting his driving attitude. It is assumed that the risk *R* consists of two components, *Re* and *Ri*[9].

$$R = Re + Ri \qquad \text{(Eq. 4-7)}$$

Re (External Risk) influences the driver's immediate reactions (e.g. there is an unexpected loss in the lighting system that causes a sudden change in luminosity, the reaction is that the driver suddenly slows down). This equates to the equilibrium between a driver's perception (e.g. he/she unexpectedly perceives the complete absence of light) and external reality (e.g., there is less light than outside), so in general it will be:

$$Re = Rre - Rp \qquad \text{(Eq. 4-8)}$$

where *Rre* is the Real External Risk and *Rp* is the Perceived Risk.

When the equilibrium exists (e.g., the failure of the lighting system is indicated by the variable message signs along the road, before entering the tunnel: the driver perceives the danger, so he slows down and pays more attention while approaching the tunnel), the reality is equal to what has been perceived:

$$Re = 0 \qquad \text{(Eq. 4-9)}$$

Therefore, *Re* acts when, after sudden changes in boundary conditions, *Rre* (External Real Risk) changes, going apart from *Rp* (perceived risk). Critical situations arise when *Rre* increases quickly in a very short time interval (e.g., if there are no warning signs approaching the tunnel, the user will be in the dark in an unexpected situation and he will be affected by hazardous reactions):

$$Re > 0 \qquad \text{(Eq. 4-10)} \qquad => \qquad Re = \delta e \qquad \text{(Eq. 4-11)}$$

Where *δe* represents the value of the risk that a driver suddenly faces (e.g. the driver continues to travel despite of low visibility). From Eq. 4-8 and Eq. 4-11:

$$\delta e = Rre - Rp \qquad \text{(Eq. 4-12)}$$

In other words, the lack of knowledge of unexpected events (e.g. the failure of the lighting system) creates a safety margin that the user can maintain with respect to the limit conditions. *Ri* (the subject's Internal Risk) affects a driver's behaviour in the medium and long term. It depends on the past experience of the driver in similar boundary conditions and on his psycho-physical conditions. It corresponds to the equilibrium between the driver's perception and his unconscious reality: in this case the word unconscious has the meaning given by Freud (1915)[10], it stands for all psychological contents that do not appear in the current horizon of consciousness. In general:

$$Ri = Rp - bS \qquad \text{(Eq. 4-13)}$$

Where *Rp* is the Perceived Risk and *bS* the Safety Budget.

When the subject must decide his "average driving attitude" according to the averagely predictable boundary conditions for a potential risk, he/she evaluates *Ri*. Obviously, it depends on the driver's condition and his/her aversion to risk.

Returning to the previous example, assuming that the driver goes on that road every day and that the lighting system often fails, he will take into account the possibility to meet the failure and will adopt a more cautious

[9] Colonna, P., & Berloco, N. (2011), "External and internal risk of the user in road safety and the necessity for a control process". *24th World Road Congress World Road Association (PIARC),* Mexico City, Mexico.

[10] Freud, S. (1915). *The unconscious*. Republished by Penguin UK in 2005.

driving behaviour. If we hypothesize that the driver pursues the safety condition, *Ri,* is null or, in case of high-risk aversion, negative. So:

$Ri \leq 0$ (Eq. 4-14) => $Ri = -\delta i$ (Eq. 4-15)

Where δi is generally a positive term, expressing the subject aversion to the risk, i.e. how much the driver is willing to invest in the Safety Budget in order to exceed the Perceived Risk, as shown in Eq. 4-13 and Eq. 4-15:

$$\delta i = bS - Rp \quad \text{(Eq. 4-16)}$$

As previously defined, δi represents a safety margin that the user keeps from his driving limit conditions, taking into account his/her past experience in similar boundary conditions and his psycho-physical condition.

Considering Eq. 4-7/8/12/13 and Eq. 4-16:

$$R = Re + Ri = Rre - Rp + Rp - bS = Rre - bS \quad \text{(Eq. 4-17)}$$

$$R = Re + Ri = \delta e - \delta i \quad \text{(Eq. 4-18)}$$

Eq. 4-17 gives the Risk as the difference between the External Real Risk and the Safety Budget; Eq. 4-18 shows that it is also equal to the difference between the Sudden Risk (that is “the value of the risk that a driver could suddenly face”) and the driver’s risk aversion.

The first impression from Eq. 4-17 and Eq. 4-18 is that the Total Risk does not depend on the Perceived Risk, but only on the External Real Risk and on the Safety Budget (i.e. on the sudden risk and the risk aversion).

This would lead to the conclusion that, in order to reduce risks, engineers should only work on factors which generate such sudden risks (road geometry, road signs, vehicle maintenance, driver’s health) or such influencing driver’s behaviour.

This would be true if the human behaviour is supposed as “algebraic”, i.e. that the sum between *Re* and *Ri* is actually correct. This is not correct, in fact *Re* and *Ri* act on human behaviour in an independent way: *Re* acts on immediate reactions in a very short time, *Ri* influences the unconscious reality. So, the resulting behaviour is the sum of different actions which can hardly influence each other.

It is necessary to consider *Re* and *Ri* separately in order to analyse the human behaviour in road safety problems, and to assess the influence of the Perceived Risk.

Therefore, we can assume that Perceived Risk in Eq. 4-7 and Eq. 4-12, related to the external risk, is different from Perceived Risk in Eq. 4-13 and Eq. 4-16, related to the Internal risk. The consequences are shown as follows:

$$Re = Rre - Rpe \quad \text{(Eq. 4-19)}$$

$$\delta e = Rre - Rpe \quad \text{(Eq. 4-20)}$$

$$Ri = Rpi - bS \quad \text{(Eq. 4-21)}$$

$$\delta i = bS - Rpi \quad \text{(Eq. 4-22)}$$

but, above all Eq. 4-18 becomes

$$R = Re + Ri = Rre - Rpe + Rpi - bS \quad \text{(Eq. 4-23)}$$

4.4.1 The external risk

It is interesting to note that the external Risk changes with the External Real Risk:

- if *Rre* suddenly increases, *Rpe* increases immediately too; if *Rpe* stays high, the driver attitude in the short-term changes (e.g. the driver slows down);
- if *Rre* decreases, before adjusting *Rp*, the driver “checks the surrounding environment”: if real conditions tend to confirm the decreasing of *Rre* for a significant period of time, after this period *Rpe* actually decreases, with a consequent modification of the subject’s behaviour and driving attitude (e.g. the driver increases his/her speed).

Rpe is modified by the boundary conditions while driving (short-term changes); it represents the natural consequence that is perceived by the user when an external factor changes. *Rpi* depends on the specific user, his/her situation and his/her experience (medium- and long-term changes).

In case of failure of the lighting system at the entrance of a tunnel, all the users perceive the change of lighting conditions, which influences *Rpe*, slowly or quickly. On the contrary, the factors that can greatly change the overall risk perception are the user's psycho-physical condition, background, experience and so the attitude towards the road. Human diversity can only be represented by a subjective variable. This also happens in the case of road safety with the *Rpi*.

Civil engineers can definitely intervene in the reduction of the external risk, introducing countermeasures. These countermeasures are effective if the risk does not become internalized by the user or if it does not induce compensations, thanks to ad-hoc intervention (like infra-red cameras to prevent high speeds). The real reason of this statement is based on the fact that the time reaction to an external risk is immediate, it happens in short time ranges. These short time ranges don't give to the driver the chance of internalizing the risk and transforming this perception into a safety budget. For example, if there is an unexpected curve, the reaction to this uncertainty is only dependant on the driver's perception. The driver has not the right amount of time to choose what behaviour should be maintained because he will instantaneously react to the external input. So, the engineering countermeasures, at least until they are not internalized or not became familiar to drivers, are reliable. After that the drivers have internalized within his/her knowledge the road layout, the external real risk would become an internal risk and so something no more related to engineering interventions but to human factors and attitudes.

So, while for the external risk engineers can have a strong positive impact on road safety performances, on the contrary the engineers may not give a significant contribution to the reduction of the internal risk.

4.4.2 The internal risk

The Internal Risk of the subject is the baseline of the Homeostatic Risk Theory, according to which the behaviour of the user who must undertake risky actions depends not only on the value of R, but also on the difference between Rpi (Perceived Risk) and bS (Safety Budget).

4.4.3 The perceived risk Rpi

When a user is about to face a potentially risky event, the risk that he perceives depends on the history that the user has lived and, on the knowledge, that the user has of events similar to the one he is approaching to.

4.4.4 The probability P

The part of p relying on human behaviour (tendency to disregard road rules) depends on both the subject condition at the considered time (health, mood, etc.) and the risk aversion.

4.4.5 The intensity I of the consequences

Human behaviour can also be influenced by the intensity of the consequences, some of them can be particularly amplified to increase the risk perceived by users, such as some measurable to change their behaviour.

For example, some enforcement measures like speed limit enforcement, the intensification of controls (safety cameras, safety tutor, etc.) or the tightening of sanctions, can be considered

The road user behaviour can therefore have detrimental consequences for him/herself if either a traffic crash occurs, or aberrant behaviours are punished.

Therefore, a road user evaluates both, the risk related to the crash, with probability Pa and consequence intensity Ia, and that relative to the punishment of aberrant behaviours, with probability Ps and intensity Is of the consequences.

"Is" therefore depends on the extent of the sanctions and, consequently, depends on the income because it may have a different effect on personal or family resources.

It follows that the amount of the penalty could be proportional to the income (as in the case of speeding fines which e.g., in Finland may be actually proportional to income).

However, physical and pecuniary damages do not represent the only negative possible consequences.

The freedom of movement is in fact a fundamental need of the man to which no one is willing to give up, so much that the limitation to freedom of movement (the prison) has always been considered by everybody as the most natural form of social and civil punishment.

On the other hand, Amartya Sen, an economist and Nobel Prize winner, clearly stated that the well-being cannot consist only in the satisfaction of material needs, since personal freedoms are essential too. Among the latter, for example, there is the freedom to act, which is closely connected to the possibility of personal movement.

It therefore follows that losing the freedom of autonomous motorized movement is a strong deterrent to disregarding rules, in general. This because it represents the threat to the limitation of a fundamental human need.

The aforementioned theories can help to explain when and why the introduction of the driving license with penalty points is effective.

4.4.6 The safety budget

Transgression also depends on the distribution of the transgression frequencies (if the majority of users are not willing to disregard rules, the result is that the few people available to this behaviour, seen as isolated and easily identifiable targets, will tend to reduce their transgression frequency – as e.g., in the Western world it has happened for smoking in public). It is therefore evident that culture modifies the bS value. Hence, a strong intervention on culture might be more effective, than other policies, in decreasing risky behaviours.

4.4.7 Operational consequences

So, the user, before making actions, unconsciously evaluates the Rpi based on personal perception of p and I, compares Rpi with the bS who is willing to spend (his own “capital of risk”, which in turn depends on risk aversion) and behaves accordingly.

- If Rpi > bS,
    - there is a significant probability that the user will modify his behaviour with respect to that initially assumed. He will do manoeuvres characterized by a lower value of p (depending on the user risk aversion) and I, until the combination of the values of p and I will lead Rpi to be again approximately equal to bS;
- if Rpi < bS,
    - it may be that the user confirms the behaviours initially assumed, adopting a prudent behaviour and maintaining the risk budget towards the perceived risk (user with high risk aversion).

An example of this situation could be provided by the situation happened in the world due to the pandemic Covid-19 in 2020. In fact, people stay at home, renouncing their freedom of movement because their safety budget is far less than the internal risk perceived, since the fear of death or other strong consequences is the deterrent to avoid risky situations. This example also suggests how the risk theory could be appropriate in several fields, not only for road safety assessments.

However, it may also happen that the user decides to use all the risk capital corresponding to his/her risk aversion, moving towards behaviours whose consequences, I, are greater, until reaching combinations of p and I values which lead Rpi to be about equal to bS.

It is also possible, somehow, to assume that the risk is the level of balance between attention and stand-by, decided by the user's will. Balance depends on the ability of the subjects to manage their neural network based on the overall level of activity or total mental workload.

Everybody therefore has a total available capital of mental workload, which depends on the original structure of his/her neural network and on its development over the years.

This mental workload capital is divided in two parts: one available for attention (linked to the desire for survival and the desire to know the meaning of one's own destiny) and the other available for the stand-by, according to a proportion defined by the level of risk naturally accepted by each person.

Over the years, the proportion changes both for natural facts (in fact, the acceptance of risk decreases with age) and for circumstances related to the life experiences of the subject (education, culture, etc.).

Basically, in human systems, homeostasis (the balance of living systems affected by constant changes) is determined by the decisions, evaluations and interactions between subjects. So, the road safety problems affected by homeostasis cannot be separated from human behavioural studies.

4.5 Road safety, railway safety, air safety

The aim of this section is to verify if the road infrastructure enhancements could: significantly increase the road safety level, be prioritized for road safety purposes.

In order to achieve this aim, it is possible to compare the crash rates of different modes of transport. Rail and air traffic have lower crash rates. Which are the reasons for this difference in crash rates?

Tab. 4.1: Crash rates of different modes of transport.

Margins of decision of different transport systems			*Mode*	*Crash Rate*
Design of the Route	*Control Systems*	*User Behaviour*		
xxxxxx	xx	xx	Railroad	xxx
xxxxxxxxx	xx	xx	Airway	x
xxxxxxxx	xxxxxxxxx	xxxxxxxxxx	Road	xxxxxxxxxx

The first column of the table states that designers have more freedom to design airways than roadways, while railway design is more influenced by the external environment than roadway design.

The second column indicates instead that the decision on where, how and when to perform traffic control by public authorities is less strict in the case of road traffic than that of rail and air traffic.

The third column, on the other hand, suggests the wider degrees of freedom of driving behaviours on roadways than on air and railways.

Finally, the last column shows, the weight of road crashes compared to rail and air crashes, even if only qualitatively.

The road crash rate is much higher than that of the other modes of transports, while the margins of variation for the design of all the investigated modes are quite similar. However, a higher degree of freedom in varying both control systems and user behaviours exists for the roadway case.

The examination of the table, even if it is a summary, immediately enables to assess that more road crashes can be caused by the lower effectiveness of the Control Systems and the User's Behaviour rather than by road design.

What about the importance of user behaviours?

From the table 4.1, it is possible to infer that the increased risk of the road system is probably due to more degree of freedoms available for the road user and because it is affected by personal behaviours. Two possible degrees of freedom are the vehicle speed and the vehicle position in the road cross section. From this point of view, it is important to highlight the behavioural difference that certainly exists between familiar users of a road section (for example commuters) and occasional users who do not know the road (for example tourists).

Tables 4.2 and 4.3 show the likely differences between the two types of users. In table 4.2, the characteristics of two different types of drivers when they face a narrow and sudden road curve are highlighted. The first type of driver is an Unfamiliar User (that is, for example, on the road for the first time) and the second one is a Familiar User.

Tab. 4.2: Behavioural differences between familiar and unfamiliar users - case 1.

Sudden narrow curve		
	Unfamiliar user	*Familiar user*
Surprise	Yes	no
Chance of being predicted	No	yes
Consequence Intensity	High	low
Need for control	Yes	no
Efficiency of control	yes/no	yes
Need for countermeasures	Yes	no
Efficiency of countermeasures	Yes	no
Possible behaviour Compensation	No	yes

The Unfamiliar User faces surprises; has no possible foreseeing situations; high intensity of consequences in case of crash; needs to be forewarned by a control system, possibly effective; requires an effective countermeasure; has no possibility to compensate his/her behaviour. For the Familiar User, the characteristics are practically opposite. In table 4.3, the same drivers' reactions are assessed in a situation in which a dog suddenly crosses the road.

Tab. 4.3: Behavioural differences between familiar and unfamiliar users - case 2.

Dog crossing the road		
	Unfamiliar user	*Familiar user*
Surprise	yes	yes
Chance of being predicted	no	no
Consequence Intensity	high	high
Need for control	no	no
Efficiency of control	no	no
Need for countermeasures	yes	yes
Efficiency of countermeasures	yes	yes
Possible behaviour Compensation	no	no

From the table it is clear that, this time, the two users behave similarly.

Comparing the two tables is useful to highlight that the behaviour of the Unfamiliar User is similar in both cases. The two tables also show that the behavioural characteristics of the Unfamiliar User are quite independent on the type of risk. Therefore, proposing a definition of a particular type of risk, the External Risk, is possible: The External Risk is a risk unknown by the driver or unforeseeable in the specific circumstances of time and place. The other risks are identified as Internal Risks.

In both the examples, the Unfamiliar User relies on the External Risk (the sudden curve is not known and the dog that crosses is not foreseeable). But the situation of the dog crossing the street (not foreseeable) represents an External Risk for the Familiar User too. Whereas, the situation of the sudden and narrow curve (that he/she knows very well) is an Internal Risk for the Familiar User.

In summary:

- the driver behaviour is different if the driver faces an External or an Internal Risk;
- it is possible to infer that the countermeasure is likely effective if:
    a) it does not foster the risk internalization
    or
    b) it fosters risk internalization without inducing compensation mechanisms related to speeds and trajectory (the two main driver's degrees of freedom).

However, when is the Behaviour Compensation impossible?

Certainly, only if the driver does not know the risk or he/she is not able to predict it and therefore only if the Risk is an External Risk. Instead, the behaviour compensation can occur if the risk is an Internal Risk.

4.6 Drivers' familiarity

4.6.1 Introduction

The final aim of road safety studies is to reduce traffic crashes, which are among the most frequent causes of death all over the world. To reach this aim, the possible causes of crashes should be known, which are several and various.

As explained in the previous sections, crashes may be generated by unexpected events. Those events consist in a difference between reality and expectations which cannot be predicted by the road user, who could have otherwise acted in order to avoid it. If the users are not able to correctly and immediately react by adapting their behaviour, then the crash will occur.

When driving, a given amount of crash risk is automatically accepted.

Crash risk-related costs increase with speeds and then, people who care about this aspect are prone to reduce their speeds. However, the speed reduction necessarily leads to both travel time and cost increasing. Hence, speed selection is strongly related to risk perception and it depends on several personal factors.

Nevertheless, since both risk perception and speed choice are strongly related to driving behaviour and since crash likelihood depends on personal differences between reality and expectations, road safety issues cannot be treated independently from driving behaviour.

Driving behaviour can be influenced by several factors, such as:

- personal data (age, gender, lifestyle, social class);
- driving experience (years of license, kilometres driven per year);
- psychological factors (elaboration of traffic inputs, ability to adapt to modified road and weather conditions, familiarity with the vehicle and the road);
- health conditions (illness, sleep, fatigue).

Each of these variables can act individually or in addition to others and contribute to define the driving behaviour.

A less frequently studied driving behavioural aspect is the drivers' familiarity with given routes and the related differences between behaviours of familiar and unfamiliar drivers. Those differences can be very important for some traffic engineering applications, even if they are very often neglected. However, those differences are crucial, since they are influential in perceiving both reality and expectations based on it, and in modifying expectations. However, those phenomena are crucial for crash occurrence. Those aspects are treated in next paragraphs.

4.6.2 Differences between familiar and unfamiliar drivers

Familiar drivers are drivers of given routes having great confidence with them, since they have frequently travelled on those routes in the past. Generally speaking, they can be commuters who travel on their daily home-to-work route.

On the other hand, unfamiliar drivers only occasionally travel on given roads. Hence, unfamiliar drivers generally travel on given routes for other reasons different than work, such as for example recreational purposes.

The different confidence with the travelled route can have a significant impact on driving behaviour. This is the reason why studying these differences is important. In fact, unfamiliar drivers assess the boundary conditions (traffic, environment, road) in real-time, while familiar drivers already know information about the road (particularly related to geometric/pavement conditions, and how the environment can influence them). Hence, familiar drivers can judge risks with fewer errors than unfamiliar ones, because they feel safer in assessing the total risk and then, their behaviour can be different due to this perception.

Some research studies confirm the above stated hypotheses. For example, Yanko and Spalek (2013)[11] have conducted a driving simulator-based experiment, by studying the behaviour of 20 drivers. They have noticed how familiar drivers (who have driven on the experimental track four times before the test) needed longer reaction times than unfamiliar drivers (who have never drove on the experimental track before) in response to unexpected

[11] Yanko M. R., Spalek, T. M. (2013), "Route familiarity breeds inattention: a driving simulator study". *Accid. Anal. and Prev.*, 57, 80-86.

external stimuli provided during the simulated driving. Martens and Fox (2007)[12] have obtained similar results from their experimental study. Those experimental results suggest that the route familiarity can lead to an increased distraction, related to the "mind wandering" phenomenon. Mind wandering means that the mind is occupied by thoughts unrelated to the current task (driving in this case) and then the answers to external stimuli can be slowed down. Theoretical expectations which suggest a reduction in the attention capacity in non-demanding driving tasks[13] (standard driving conditions, corresponding to minimum waste of drivers' mental energies) are in accordance with the above presented results. The decreased attention and the perceptual difference can result in more dangerous behaviours (more traffic violations and higher speeds) for familiar drivers, as highlighted by e.g. Rosenbloom et al. (2007)[14].

The behavioural differences in perception and attention between familiar and unfamiliar drivers can influence the measure of road safety-related parameters such as drivers' speeds and trajectories. Those two variables can be considered as measurable outputs of a driving behaviour influenced by the above described human factors, and by route familiarity among them. The influence of differences between familiar and unfamiliar drivers on speeds and trajectories, as well as the practical implications for road and traffic engineering, are discussed as follows in next paragraphs.

However, besides aspects of importance for the road and traffic engineering practice, the definition of "drivers' familiarity" for experimental and practical purposes is extremely blurry. Hence, some indications on how to possibly measure and define the drivers' route familiarity are given as follows.

4.6.3 Measures of drivers' route familiarity

Measuring and identifying the drivers' familiarity is essential for experimental and practical purposes in the field of road safety. Safety research inquiring into behavioural aspects and, in particular, drivers' familiarity, can be conducted in different ways (Intini et al., 2019[15]):

- through direct observations (on-road or driving simulator-based study);
- through indirect observations based on descriptive and statistical analysis on crash datasets;
- through indirect observations based on results from surveys.

In direct observation studies, the behaviour of familiar drivers (who, for instance, have driven several times in the past on the same route) is compared to that of unfamiliar drivers. In particular, the effects of the familiarization process can be observed on the same driver (i.e., by measuring the speed changes, see Colonna et al. (2016)[16], or the changes in the response to external stimuli, see Martens and Fox, 2007[12]) or by comparing behaviours of familiar and unfamiliar drivers with the same scenarios (see e.g., Yanko and Spalek, 2013[11]). Alternatively, the behaviour of drivers already familiar with a given condition can be compared with the same drivers in unfamiliar conditions (see e.g., Rosenbloom et al., 2007[14]). However, in studies in which the familiarization process was observed, different numbers of repetitions were used. The problem is to define how many repetitions are sufficient to consider a driver as "familiar" with a given road condition.

In most studies based on crash datasets, driver- and crash-related attributes and circumstances of crashes occurred beyond and within given distances from the residence are compared. Even in this case, the distance threshold to consider a driver as unfamiliar is hard to set. Very often crashes occurred to foreign drivers are compared to crashes of residents (e.g., Yannis et al., 2007[17]) or alternatively, the involvement or rural/urban residents in rural/urban crashes is analysed (Blatt and Furman, 1998[18]). The absolute distance in kilometres from

[12] Martens M. H., Fox M. R. J. (2007), "Do familiarity and expectations change perception? Drivers' glances and response to changes". *Transportation Research Part F: Traffic Psychology and Behaviour,* 10, 476-492.

[13] Young M., Stanton N. (2002). "Malleable attentional resources theory: a new explanation for the effects of mental underload on performance", *Human Factors,* 44(3), 365-375.

[14] Rosenbloom T., Perlman A., Shahar A. (2007), "Women drivers' behavior in well-known versus less familiar locations", *Journal of Safety Research*, 38(3), 283-288.

[15] Intini P., Colonna P., Ryeng E. O. (2019), "Route familiarity in road safety: A literature review and an identification proposal", *Transportation research part F: traffic psychology and behaviour*, 62, 651-671.

[16] Colonna P., Intini P., Berloco N., Ranieri V. (2016), "The influence of memory on driving behavior: How route familiarity is related to speed choice. An on-road study", *Safety Science*, 82, 456-468.

[17] Yannis G., Golias J., Papadimitriou E. (2007), "Accident risk of foreign drivers in various road environments", *Journal of safety research*, 38(4), 471-480.

[18] Blatt J., Furman S. M. (1998), "Residence location of drivers involved in fatal crashes", *Accident Analysis & Prevention*, 30(6), 705-711.

residence can be also used as a variable for relating crash likelihood and characteristics to familiarity (Intini et al., 2018[19]).

In survey-based studies, drivers were directly asked about their familiarity with the road on which crashes occurred or to which the other study variables are computed. However, also in this case, different studies have set different minimum traveling frequencies for identifying the drivers' familiarity (e.g., from daily, see Baldock et al., 2005[20], to at least biannually, see Ryeng, 2012[21]).

Hence, in the studies performed, frequency-based (mostly in observational and survey-based studies) or distance-based scales (mostly in crash database analyses) were used to measure and identify the drivers' familiarity. In any case, a certain arbitrariness in the thresholds used is evident, which reflects the difficulty in catching differences which are not deterministic but are strictly related to the human behaviour.

However, to reveal the relationships between familiarity and safety-related variables for research purposes, some indicators should be defined. The use of a standardized procedure, even if potentially biased for the reasons explained above, may ensure comparability between results from different studies. The specific identification criteria proposed in Intini, et al. (2019)[15] are shown in Table 4.4 for both frequency and distance-based scales. They are calibrated as based on an extensive review of studies directly or indirectly focused on the study of drivers' familiarity. In fact, most of previous studies reviewed are entirely or partially coherent with this proposal.

Tab. 4.4: Proposed frequency and distance-based identification criteria for drivers' route familiarity (taken from Intini et al., 2019[15]).

Scale	*Familiarity*	*Unfamiliarity*
Frequency-based	Survey-based: Frequency ≥ Weekly Experimental studies: Test repetitions ≥ (4-7)	Survey-based: Frequency ≤ Yearly Experimental studies: First time driving (besides practice)
Distance-based	Distance ≤ locally calibrated average commuting trip distance	Distance ≥ locally calibrated minimum long-trip distance (foreigners = unfamiliar)

The reasons for the frequency measures in the previous table are as follows:

- drivers can be familiar with a route even if not strictly daily travelled (such as commuters). In fact, some behavioural changes were observed even after four/five subsequent traveling days (see e.g., Colonna et al., 2016[16]) or 4 test repetitions in the same day (Yanko and Spalek, 2013[11]). Moreover, interpreting the results obtained from Li et al. (2018)[22], more than 7 test repetitions over
- different days may be not necessary (performances may be stabilized). In survey-based studies, the familiar threshold should be at least weekly-based, to increase the opportunity of keeping enough memories from road scenes (to avoid possible comeback to the unfamiliarity performance, see Colonna et al., 2016[16]).
- A driver who never travelled a given route is unfamiliar with it. However, also drivers who travelled rarely on a given route can be considered as familiar with it. On the other hand, possible long-term memory effects can occur (Rankin et al., 2009[23]), as experimentally observed after almost one month (Colonna et al., 2016[16]). Hence, a yearly frequency is suggested as a minimum threshold for defining drivers as unfamiliar.

The reasons for the distance measures in the previous table are as follows:

Drivers spend several travels on roads near residence (typically commuting routes), being familiar with them. Hence, a possible definition is based on the drivers' commuting distance, which may vary within countries. This

[19] Intini P., Berloco N., Colonna P., Ranieri V., Ryeng E. (2018), "Exploring the relationships between drivers' familiarity and two-lane rural road accidents. A multi-level study", *Accident Analysis & Prevention*, 111, 280-296.

[20] Baldock M. R. J., Long A. D., Lindsay V. L. A., McLean, J. (2005), *Rear end crashes*, University of Adelaide. Centre for Automotive Safety Research.

[21] Ryeng E. (2012), "The effect of sanctions and police enforcement on drivers' choice of speed", *Accident Analysis & Prevention,* 45, 446-454.

[22] Li X., Li Z., Cao Z., Zhao X. (2018), "Modeling drivers' memory of daily repetitive stimuli in traffic scenes", *Cognition, Technology & Work,* 20(3), 389-399.

[23] Rankin C. H., Abrams T., Barry R. J., Bhatnagar S., Clayton D. F., Colombo J., Coppola G., Geyer M. A., Glanzman, D. L., Marsland S., McSweeney F. K., Wilson D. A., Wu C. F., Thompson R. F. (2009), "Habituation revisited: an updated and revised description of the behavioural characteristics of habituation", *Neurobiology of learning and memory*, 92(2), 135-138.

distance measure is more flexible than other arbitrary limits (town or state limits). In fact, drivers may be unfamiliar with roads in the same country but very far from residence.

Drivers are often unfamiliar with roads very far from residence. Long trips are rarer than commuting trips and for then, other transport modes can be chosen for them. Hence, the unfamiliarity threshold can be set according to the average distance (which may vary in different countries) above which other means of transport are preferred to car. Moreover, based on previous studies, foreign drivers should be anyway considered as unfamiliar (Yannis et al., 2007[17]), because they often do not know road scenarios of different countries. Frequencies between weekly and yearly and distances between average commuting and long trips can represent “transition” drivers, not classifiable neither as familiar nor as unfamiliar with a reasonable margin of error.

4.6.4 Route familiarity issues in road and traffic engineering

In this paragraph, the influence of drivers’ route familiarity in the traffic engineering practice is described.

The calculation of Levels of Service (LOS) is aimed at quantifying the quality of road service, in order to meet some adequate standards for road infrastructures. In the American traffic engineering handbook Highway Capacity Manual (2010)[24], levels of service are computed as a function of average speed and traffic flow. In particular, the flow rate is computed through the following equation:

$$Vp = \frac{V}{PHF*N*f_{HV}*f_p} \qquad \text{(Eq. 4-24)}$$

Where:

- Vp = equivalent flow of passenger cars for a 15-minutes period (passenger vehicles/hour/lane),
- V = peak hour traffic (vehicles/hour),
- PHF = Peak Hour Factor,
- N= number of lanes,
- fHV = heavy vehicle factor,
- fp = driver population factor.

The introduction of the fp factor in the equivalent flow rate calculation allows considering the different composition of the driver population as based on their route familiarity and dividing them in two main categories.

Regular users (route familiar): all drivers who travel with high frequency (almost daily) on the considered route, such as for example commuters.

Non-regular drivers (route unfamiliar): all drivers who travel with low frequency on the considered route, such as for example tourists, recreational drivers, or anyway non-commuters.

HCM 2010 suggests setting fp to 1 in case of traffic flow mainly composed of regular users and between 0.85 and 1 in case of a notable presence of non-regular users. However, the fp decrease down to 0.85 is theoretically linked to an increase of Vp up to 20% more than the same value computed for fp equal to 1. For uninterrupted facilities, the increase in the equivalent flow rate is proportional to an increase in the density of equivalent passenger cars per km and then, to a level of service worsening. Hence, the HCM 2010 framework provides that the presence of non-regular drivers on uninterrupted facilities such as main rural roads and freeways can be directly related to a level of service worsening of the road itself. This concept is graphically reported in next figure. In the figure, an increase in the equivalent flow rate due to the introduction of a fp value less than 1 (relevant presence of non-regular drivers) is highlighted for different values of free flow speeds. The equivalent flow rate increase is related to a decrease in the average traffic speed and to a consequent level of service worsening (e.g., in the example figure, from LOS C to D, or from LOS D to E, depending on the initial speed). Hence, the traffic could be slowed down by the presence of non-regular drivers (i.e., unfamiliar with the route), as indicated by the decrease in the average traffic speed and this effect increases with the initial traffic free flow speed increasing.

[24] AASHTO (2010), *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington D.C., USA.

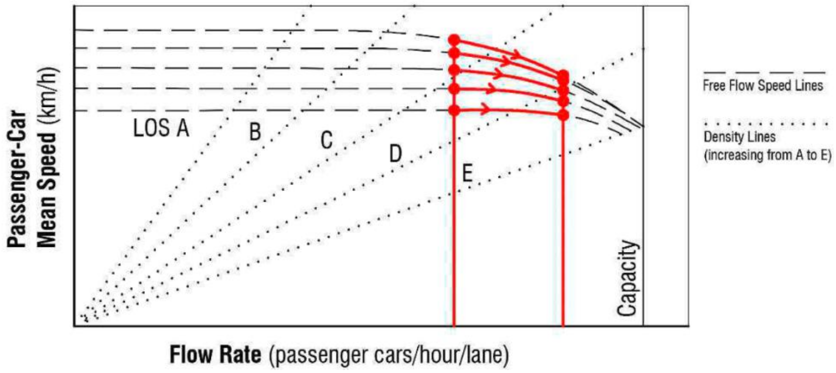

Fig. 4.2: Example of effects of increasing the equivalent flow rate on the level of service (taken from Intini, 2017[25], based on the Highway Capacity Manual, 2010[24]).

It is important to note that in the most recent version of the Highway Capacity Manual (2016)[26], the presence of recreational drivers is directly considered through a capacity reduction factor rather than using an equivalent flow rate increasing factor (fp). Even if the approach is different, a worsening in the road serviceability is expected in this case as well.

On the other hand, a common recommendation in road design guidelines is to design roads with particular regard to unfamiliar drivers. In fact, the road alignment should be thought for drivers who travel for the first time on the route and they have no familiarity with it[27]. This strategy should avoid high differences between reality and expectations in sections of the road layout in which the sequence of road geometric elements is particularly demanding. The latter condition can be extremely dangerous in particular for route unfamiliar drivers (non-regular users).

However, the behaviour of familiar drivers should be taken into account as well, and their tendency to inattention, distraction and dangerous driving, previously discussed. Hence, even if on one hand the road should be designed in order to minimize differences between reality and expectations by preserving unfamiliar drivers, on the other hand habitude effects in familiar drivers should be avoided as well. In fact, the already likely inattention and disregard of road rules could be eased.

Finally, the drivers' familiarity and, in particular, the differences between familiar and unfamiliar drivers have a twofold explanation in road and traffic engineering. Both drivers' categories should be preserved while designing roads (by avoiding unexpected changes for unfamiliar drivers and monotony for familiar drivers). However, in parallel, the negative interactions between the two categories of drivers with respect to traffic serviceability in uninterrupted flow conditions (average speed/capacity decrease and level of service worsening) should be considered as well.

A practical application showing the meaning of these aspects is discussed in the final paragraph of this section.

4.6.5 Influence of familiarity on drivers' speeds and trajectories

Differences between familiar and unfamiliar drivers can be measured, as previously stated, by considering the typical outputs of the driving process, such as speed and lateral position. In this paragraph, some experimental results from research studies are presented. They concern the assessment of behavioural changes, in terms of both speeds and trajectories (lateral position), once drivers have become familiar with a given route. The first evident result is related to speed choice. Route familiar drivers select higher speeds, on average, than drivers unfamiliar with the same route[16]. This result comes from an on-road experiment in which a sample of drivers have travelled on a two-way two-lane very-low volume rural road six times in six different days (four subsequent days and the other two days more distant in time). Drivers' speeds and trajectories have been monitored and recorded through GPS technology. In this way, it was possible to note how unfamiliar drivers' speeds increased over days until a kind of asymptote was reached. This almost constant level may correspond to the speed chosen while drivers

[25] Intini P. (2017), *Route familiarity in road safety: Theory and applications*, Doctoral thesis. Polytechnic University of Bari, Bari, Italy.

[26] AASHTO (2016), *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington D.C., USA.

[27] Milliken J. G., Council F. M., Gainer T. W., Garber N. J., Gebbie, K. M., Hall, J. W., et al. (1998), "Managing speed: review of current practice for setting and enforcing speed limits", *Transportation Research Board, Special Report 254*, Washington DC.

have become route familiar (see next figure). This result is in accordance with the HCM methodology for traffic flow calculations. In fact, as previously discussed, a noticeable share of unfamiliar drivers in the traffic flow could be related to an average traffic speed decrease. The speed decreasing could be associated, ceteris paribus, to the lower average speed selected by unfamiliar drivers on the considered road. Furthermore, the experimentally found speed increasing is not greatly dependent on road geometry: familiar drivers select, on average, higher speeds than the unfamiliar drivers on all the road segments analysed. In particular, considering sight distance as a main road geometric variable, the speed difference between the unfamiliarity condition (first day test) and the acquired familiarity condition (fourth day test) can be noted in all visibility conditions (based on the available sight distance).

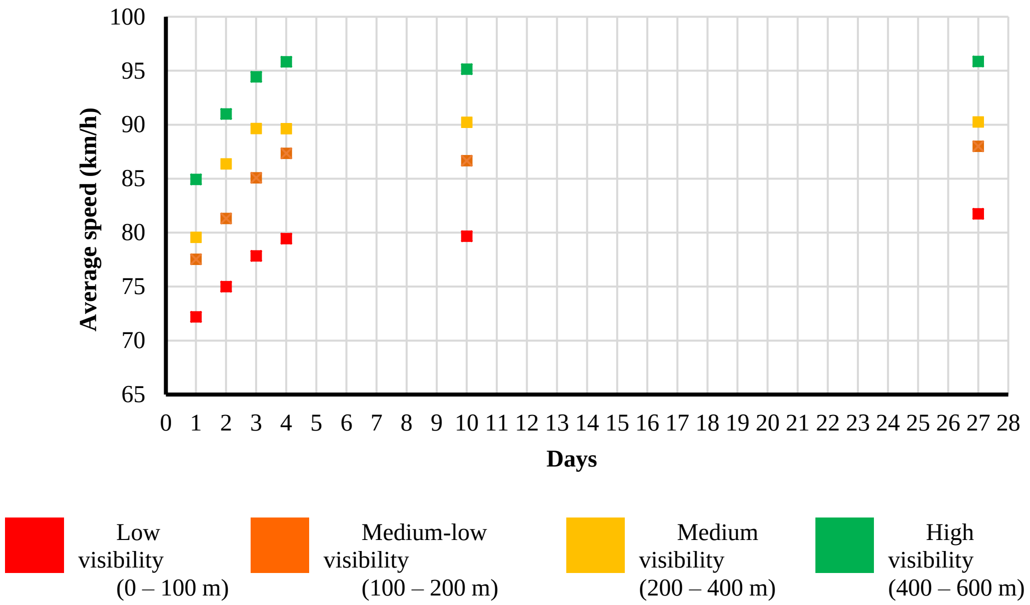

Fig. 4.3: Speed differences between drivers who have acquired route familiarity (fourth day test) and drivers who are initially unfamiliar (first day test) – Results for different visibility classes (based on Colonna et al., 2016[16]).

As another speed-related result, familiar drivers were noted to be more prone to underestimating driving risks, with respect to unfamiliar drivers. This finding was based on another part of the same previously described experimental test. In fact, it was asked to the same 20 drivers to travel on the same experimental route under four different speed tasks: free speed driving task, low speed driving task, medium speed driving task, and a high-speed driving task. Results from this further experiment (see next figure) suggest that drivers who have acquired familiarity with the route (after four driving tests in four subsequent days) tend to freely choose (in the free speed test) speeds closer to those considered high by themselves with respect to the first test day. In fact, in the first test day, the average free speed was close to that considered as a medium speed instead[28]. This phenomenon could be related to the previously discussed differences between familiar and unfamiliar drivers: familiar drivers may tend, on average, to underestimate driving risks on the same route. However, the underestimation tendency may be partly unconscious, since familiar drivers may progressively switch to a sort of automated driving pattern[29].

[28] Colonna P., Intini P., Berloco N., Ranieri V. (2015), "Route familiarity in road safety: speed choice and risk perception based on an on-road study", *Compendium of Papers of the 84th Annual Meeting of the Transportation Research Board,* Washington DC, USA.
[29] Charlton S. G., Starkey N. J. (2013), "Driving on familiar roads: Automaticity and inattention blindness", *Transportation research part F: traffic psychology and behaviour*, 19, 121-133.

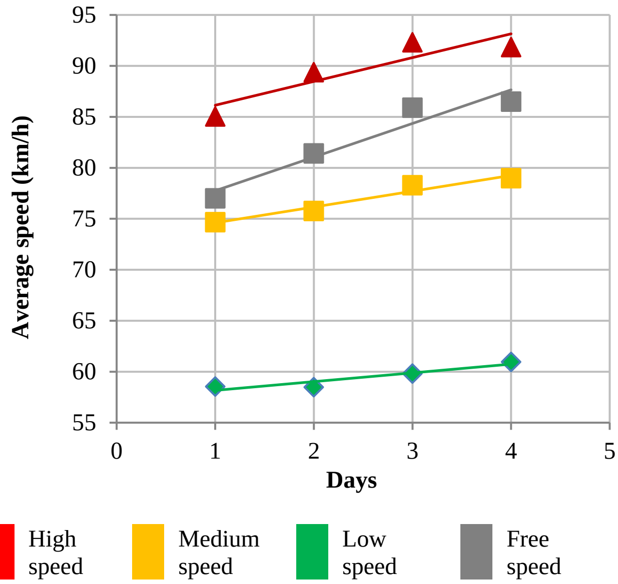

Fig. 4.4: Speed differences between drivers who have acquired route familiarity (fourth day test) and drivers who are initially unfamiliar (first day test) – Results for different speed tasks (based on Colonna et al, 2016 [16]).

Another driving behaviour-related measurable variable which has resulted to be influenced by drivers' route familiarity is the curve trajectory. In fact, the average radius of curve trajectories followed by familiar drivers was higher, on average, than the radius followed by unfamiliar drivers.[30] In particular, this difference is greater in curves having large radii, as shown in next figure. In this case, high degrees of freedom in choosing speeds and trajectories can be likely considered. This result can be interpreted in parallel with the discussed speed-related finding: a speed increase for familiar drivers is clearly related to an increase in the radii of curve trajectories, in order to avoid skidding in curves. However, the side effect of this process is the increased curve cutting tendency, and the encroachments in the opposite lane, which are both dangerous behaviours. Moreover, these aberrant behaviours are already present in the first test day, in which radii of curve trajectories were considerably higher than the curve radius.

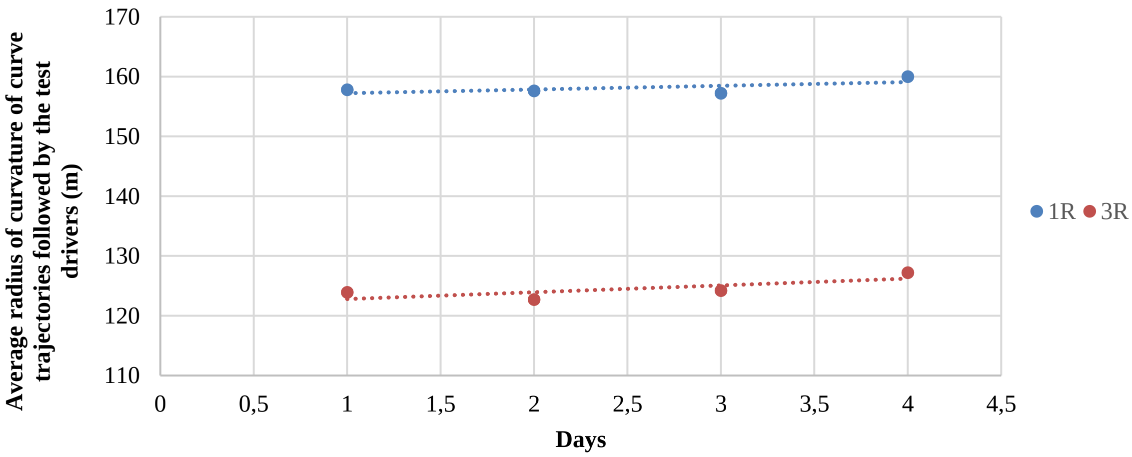

Fig. 4.5: Differences in the radii of curve trajectories between drivers who have acquired route familiarity (fourth day test) and drivers who are initially unfamiliar (first day test) – Results for two test curves (based on Colonna et al., 2016[16]).

[30] Colonna P., Intini P., Berloco N., Perruccio A., Ranieri V. (2016), "Repeated measurements of lateral position and speed at horizontal curves on a very-low volume rural road", *Compendium of Papers of the 85th Annual Meeting of the Transportation Research Board*, Washington DC, USA.

4.6.6 Influence of familiarity on road crashes

In this paragraph, results from previous studies in which drivers' familiarity was related to road crashes (both likelihood and types) are summarized. Firstly, it should be stated that research is not abundant on this topic (considering also the difficulty of inquiring into familiarity aspects based on road crashes, as previously discussed).

As far as the accident occurrence is concerned, being familiar with the route was identified as a crash factor by Brown et al. (2015)[31], in their Australian case-control study focused on bikers (daily drivers against drivers who never travelled on the crash site). Moreover, Blatt and Furman (1998)[18] found an over-involvement of rural residents in fatal crashes on rural roads and the same effect for urban residents in urban crashes. This effect was confirmed by Donaldson A.E. et al. (2006)[32], who however highlighted that urban drivers have the highest risk of fatality when involved in rural crashes (rather than urban). On the other hand, being unfamiliar with the road (a foreign driver in particular) was identified as a crash factor as well. This effect was found by both Yannis et al. (2007)[17] and Kim et al. (2012)[33], who found foreign drivers to be more likely at-fault in crashes. In the first study, foreign drivers (from EU) were found to have a higher accident risk at junctions in Greece while, in the second study, visitors were found to be more likely involved in improper manoeuvre or wrong way crashes than Hawaii residents.

For what concerns accident types instead, familiar drivers (daily to monthly drivers) were found to be more likely to be involved in run-off crashes than other drivers (by Liu and Ye, 2011[34]). Concerning rear-end crashes, daily drivers were found to be more likely struck than striking (Baldock et al., 2005[20]). On the other hand, International drivers were found to be over-represented in angle, sideswipe and head-on collisions than resident drivers, who were found to be over-represented in fixed-object, pedestrians, parked vehicle and animal collisions instead (Wilks et al., 1999[35]).

Some of the previously cited studies were not specifically dedicated to studying the relationships between drivers' familiarity and road crashes. A detailed study on this topic was recently made by Intini et al. (2018)[19], by relying on the previously defined distance-based familiarity identification criterion (calculating distances from zip codes of the drivers' residence in a Norwegian crash dataset). They used an integrated approach composed of a multi-level analysis, including:

- a macro-analysis for comparing accident rates at different road sites according to the seasonal variation of the average daily traffic (assuming that a relevant share of unfamiliar drivers such as tourists may be responsible for the seasonal traffic increasing);
- a detailed statistical analysis based on logistic regression models for finding relationships between drivers' familiarity with road sites on which they crashed and the accident characteristics;
- a micro-analysis aimed at finding specific associations between drivers' familiarity and accident types and circumstances, also considering the interactions between different drivers.

The results from the study, which was focused on two-way two-lane Norwegian rural roads can be summarized as follows:

the macro indicator accident rate was not revealed to be useful in identifying the specific relationship between drivers' familiarity and the crash occurrence. This can be explained by the several other factors explaining the accident rate besides drivers' familiarity.

The further detailed analyses performed have confirmed the familiarity as an influential factor for crash risk. In particular, crashes to unfamiliar drivers were associated to sites with high summer traffic variation during summer months. Crashes to familiar drivers were possibly associated to distraction and aberrant behaviours due to over-confidence.

[31] Brown J., Fitzharris M., Baldock M., Albanese B., Meredith L., Whyte T., Oomens M., (2015), *Austroads, Sidney. Motorcycle In-depth Crash Study* (No. AP-R489-15)

[32] Donaldson A. E., Cook L. J., Hutchings C. B., Dean J. M. (2006), "Crossing county lines: the impact of crash location and driver's residence on motor vehicle crash fatality", *Accid. Anal. Prev.,* 38 (4), 723-727.

[33] Kim K., Brunner I. M., Yanashita E, Uyeno R. (2012), "Comparative assessment of visitor and resident crash risk in Hawaii", *Transportation Research Board 91 St Annual Meeting* 12-2854.

[34] Liu C., Ye T. J., (2011), *Run-off-road Crashes: An On-scene Perspective* (No. HS-811 500).

[35] Wilks J., Watson B.C., Johnston K.L., Hansen J.A. (1999), "International drivers in unfamiliar surroundings: the problem of disorientation", *Travel Med. Int.,* 17(6), 162-167.

Some associations between drivers' familiarity and specific crash types and circumstances were suggested (e.g, familiar drivers with rear-end crashes, see also Intini et al., 2017[36]). However, some other aspects are less clear, such as the role of interactions between familiar and unfamiliar drivers.

In another study by Intini et al. (2019)[37], the same distance-based strategy was used, to identify drivers' familiarity as based on zip codes from crash datasets. However, this study was focused on finding evidences of the relationships between familiarity of drivers crashed at given road sites and the geometric characteristics of these sites, since inconsistencies in the road horizontal and vertical alignments may cause dangerous unexpected road scenarios.

In particular, run-off-road single-vehicle crashes at rural road curves on Norwegian two-way two-lane rural roads were investigated. Logistic regression modelling was used, having familiarity as a binary dependent variable (familiar/unfamiliar crashed driver) and road consistency variables as predictors.

The following results from this study can be highlighted:

familiar drivers were more associated to dangerous behaviours (e.g., higher speeds) than unfamiliar drivers, in particular in highly curved road sections with large curvature variations.

Unfamiliar drivers were found to particularly crash at curves with an unexpected radius (radius much smaller than the previous curve), unexpected length (length much greater than the previous curve), and at horizontal curves combined with vertical curves possibly leading to increased workload and recognition issues.

Based on these results and on the dataset analysed, some possible practical consequences for road design were suggested, specifically related to:

the consistency of subsequent curve radii, by proposing stricter admissibility areas for roads mainly travelled by unfamiliar drivers, such as touristic routes;

the coordination of vertical and horizontal curves, by suggesting a minimum requirement based on the crash dataset investigated.

The proposed requirement for the coordination of vertical and horizontal curves on roads frequently travelled by unfamiliar drivers (e.g., tourists) is reported as follows:

$$\frac{Rv}{Rh} > 23 \div 30 \qquad \text{(Eq. 4-25)}$$

Where Rh is the radius of the horizontal curve and Rv is the radius of the vertical curve, placed in the same road section.

While the efforts to reveal relationships between drivers' familiarity and road crashes were presented, it is evident how clear findings are hard to be obtained from crash datasets only. Further studies are needed to address this matter, possibly using other strategies, such as surveys (see e.g., Intini et al., 2020[38]) or observations based on naturalistic driving studies (Wu and Xu, 2018[39]).

4.6.7 Practical application

In this paragraph, a possible example of engineering application concerning the previously explained theoretical frameworks and experimental findings is reported.

A double carriageway rural road, two-way operated, with two lanes per direction is assumed. After, different scenarios are built, considering different distributions of the driver population (familiar and unfamiliar) on the same road. The concepts exposed in the HCM (reference: 2010)[24] for the calculation of the equivalent flow rate (Eq. 4-24) are applied. No precise indications on how to select the fp factor in the Eq. 4-24 are given and then, the following equivalence table is assumed.

[36] Intini P., Colonna P., Berloco N., Ranieri V., Ryeng E. (2017), "The relationships between familiarity and road accidents: Some case studies*", Transport Infrastructure and Systems: Proceedings of the AIIT International Congress on Transport Infrastructure and Systems,* Rome, Italy.

[37] Intini P., Berloco N., Colonna P., Ottersland Granås S., Olaussen Ryeng E. (2019), "Influence of road geometric design consistency on familiar and unfamiliar drivers' performances: crash-based analysis", *Transportation research record,* 2673(10), 489-500.

[38] Intini P., Berloco N., Colonna P., De Gennaro D., Ranieri V., Ryeng E. (2020), "Self-Reported Route Familiarity and Road Safety Negative Outcomes: First Results from a Transnational Survey-Based Study", *Transportation Research Procedia,* 45, 46-53.

[39] Wu J., Xu H. (2018), "The influence of road familiarity on distracted driving activities and driving operation using naturalistic driving study data*", Transportation research part F: traffic psychology and behavior,* 52, 75-85.

Tab. 4.5: Conversion of unfamiliar drivers' percentage in the traffic flow into the driver population factor fp (in the range considered by the HCM, 2010[24]).

Unfamiliar drivers' percentage in the traffic flow (%)	*Driver population factor fp (-)*
0	1.00
10	0.95
20	0.90
30	0.85

The equivalent flow rate is calculated as based on Eq. 4-6 (recalled as follows) for different fp values in the previous table (0.85-1.00) and different flow rates V, assuming the other factors (PHF, f_{HV}) equal to 1.

$$Vp = \frac{V}{PHF*N*f_{HV}*f_p} \qquad \text{(Eq. 4-26)}$$

Given the example road type considered, the free flow speed can be set to 100 km/h, which is related to a capacity of 2200 vehicles/hour/lane. Vp values are increased with respect to V values, in case of fp factors smaller than 1.

The increase in the equivalent flow rate is related to a reduction in the average traffic flow speed (see Fig. 4.2). This reduction can be attributed to the presence of unfamiliar drivers in the traffic flow, reasonably assuming that they may travel at lower speeds, such as experimentally demonstrated. It can be empirically assumed that the two categories of drivers may have different average speeds and that the average traffic speed can be the weighted mean of the two speeds, as based on the relative percentages of familiar and unfamiliar drivers in the traffic flow. This concept is summarized in the following equation:

$$V_{average,TF} = V_{average,UFCF} * x + V_{average,FCF} * (1 - x) \qquad \text{(Eq. 4-27)}$$

Where:

- $V_{average,TF}$= Average speed of the traffic flow,
- $V_{average,UF}$= Average speed of the unfamiliar drivers' share in the traffic flow,
- $V_{average,F}$= Average speed of the familiar drivers' share in the traffic flow,
- x= unfamiliar drivers' share in the traffic flow.

The above reported equation can be used to achieve the average speed of the unfamiliar share of drivers in the traffic flow, assuming that the average speed of the familiar share of drivers is equal to that computed in case of fp=1 (for each V value), that is assuming that there are no unfamiliar drivers.

If the computed speed of the share of unfamiliar drivers is smaller than that corresponding to capacity (which is an unlikely event), the process is inverted. This means that the speed corresponding to capacity is assigned to the unfamiliar share of drivers and the familiar speed is computed from Eq. 4-27, since the average traffic speed is known.

The application of this procedure leads to calculating the differences between the average speed of familiar drivers and that of unfamiliar drivers (Δspeed), for each flow V (in this example, from 1300 to 2200 vehicles/hour/lane) and each fp coefficient (from 0.85 to 1.00). The calculations of the above described procedure and the speed differences obtained are reported in the previous table.

Following this simple empirical procedure based on the HCM, the average speed differences between familiar and unfamiliar drivers can be simulated. In some cases, they are greater than 10 km/h. Even if experimental studies are needed to confirm the result from this simulation, it is reasonable and realistic to think that, independently on precise speed differences, a relevant share of unfamiliar drivers in the traffic flow leads to the speed variance increasing (scarce homogeneity among drivers' speed choice).

Since the speed variance can be directly linked to the crash risk (through a crash rate-speed variance power law[40]), the high-speed variability in the traffic flow should be generally limited. Concerning the differences between familiar and unfamiliar drivers, the speed differences emerging in case of a relevant presence of unfamiliar drivers should be limited. Hence, for example, variable speed limits can be implemented, to be lowered in given hours, days or year periods, in which a relevant presence of route unfamiliar drivers (e.g. a noticeable touristic flow) is expected.

[40] Garber N. J., Gadiraju R. (1989), "Factors affecting speed variance and its influence on accidents", *Transportation Research Record,* 1213, 64-71.

Tab. 4.6: Calculation of the average speed difference between familiar and unfamiliar shares of drivers for different values of V and different percentages of unfamiliar drivers in the flow (FFS = 110 km/h, capacity = 2200 vehicles/hour/lane) (based on Intini, 2017[25]).

V (vehi/ lane)	*fp*	*% Not regular users*	*Vp (vehi/ lane)*	*Vp[1] (vehi/ lane)*	*Average flow speed (km/h)*	*Average not regular users' speed (km/h)*	*Average not regular users' speed[2] (km/h)*	*Users' speed - Not regular users' speed (km/h)*	*Average users' speed (km/h)*
1300	1.00	0.00	1300	1300	100.0	-	-	-	100.0
1300	0.95	0.01	1368	1368	100.0	100.0	100.0	0.0	100.0
1300	0.90	0.02	1444	1444	99.7	98.6	98.6	1.4	100.0
1300	0.85	0.03	1529	1529	98.9	96.6	96.6	3.7	100.0
1400	1.00	0.00	1400	1400	100.0	-	-	-	100.0
1400	0.95	0.01	1474	1474	99.5	94.7	94.7	5.3	100.0
1400	0.90	0.02	1556	1556	98.6	93.0	93.0	7.0	100.0
1400	0.85	0.03	1647	1647	97.4	91.4	91.4	8.6	100.0
1500	1.00	0.00	1500	1500	99.2	-	-	-	99.2
1500	0.95	0.01	1579	1579	98.3	90.2	90.2	9.0	99.2
1500	0.90	0.02	1667	1667	97.2	88.9	88.9	10.3	99.2
1500	0.85	0.03	1765	1765	95.7	87.5	88.0	11.0	99.0
1600	1.00	0.00	1600	1600	98.0	-	-	-	98.0
1600	0.95	0.01	1684	1684	96.9	86.6	88.0	9.9	97.9
1600	0.90	0.02	1778	1778	95.5	85.4	88.0	9.4	97.4
1600	0.85	0.03	1882	1882	93.8	83.9	88.0	8.3	96.3
1700	1.00	0.00	1700	1700	96.7	-	-	-	96.7
1700	0.95	0.01	1789	1789	95.3	83.1	88.0	8.1	96.1
1700	0.90	0.02	1889	1889	93.7	81.8	88.0	7.1	95.1
1700	0.85	0.03	2000	2000	91.8	80.3	88.0	5.4	93.4
1800	1.00	0.00	1800	1800	95.2	-	-	-	95.2
1800	0.95	0.01	1895	1895	93.6	79.6	88.0	6.2	94.2
1800	0.90	0.02	2000	2000	91.8	78.2	88.0	4.7	92.7
1800	0.85	0.03	2118	2118	89.6	76.6	88.0	2.3	90.3
1900	1.00	0.00	1900	1900	93.5	-	-	-	93.5
1900	0.95	0.01	2000	2000	91.8	76.0	88.0	4.2	92.2
1900	0.90	0.02	2111	2111	89.7	74.5	88.0	2.1	90.1
1900	0.85	0.03	2235	2200	88.0	75.1	88.0	0.0	88.0
20000	1.00	0.00	2000	2000	91.8	-	-	-	91.8
20000	0.95	0.01	2105	2105	89.8	72.4	88.0	2.0	90.0
20000	0.90	0.02	2222	2200	88.0	72.9	88.0	0.0	88.0
20000	0.85	0.03	2353	2200	88.0	79.2	88.0	0.0	88.0
21000	1.00	0.00	2100	2100	89.9	-	-	-	89.0
21000	0.95	0.01	2211	2200	88.0	70.7	88.0	0.0	88.0
21000	0.90	0.02	2333	2200	88.0	80.3	88.0	0.0	88.0
21000	0.85	0.03	2471	2200	88.0	83.5	88.0	0.0	88.0
22000	1.00	0.00	2200	2200	88.0	-	-	-	88.0
22000	0.95	0.01	2316	2200	88.0	88.0	88.0	0.0	88.0
22000	0.90	0.02	2444	2200	88.0	88.0	88.0	0.0	88.0
22000	0.85	0.03	2588	2200	88.0	88.0	88.0	0.0	88.0

[1]Corrected by limiting the value to the capacity (2200 vehicles/hour/lane).
[2]Corrected by limiting the value to the minimum speed at capacity (88 km/h).

Based on the simulations performed and presented, the speed limit could be e.g., lowered by 10 km/h (which roughly corresponds to the maximum speed difference between the two categories of drivers), in case of relevant presence of unfamiliar drivers on four-lane double-carriageway rural roads, having free flow speed of 100 km/h. The same procedure could be repeated for different speeds, in order to find lower speeds which can limit the speed variance among the two categories of drivers.

4.6.8 Conclusions

Differences between route familiar and unfamiliar drivers are influential from a twofold perspective. In fact, they act both at an individual level by modifying the drivers' perception and their behaviour, and at a reciprocal interaction level, as discussed in the previous paragraphs.

Route familiar drivers can modify their behaviour with respect to the unfamiliar drivers by selecting higher speeds, travelling in curves with larger radii of curve trajectories, underestimating the driving risks and being more prone to inattention and distraction.

On the other hand, unfamiliar drivers could make driving errors because they do not know the road, since the road reality is much more different than the expectations.

Hence, in the road engineering practice, both tendencies should be considered through a road design process which should avoid both excessive complexity and monotony. Furthermore, issues related to the interactions between the two categories of drivers should be considered such as indicated e.g., in the application previously described (variable speed limits can be implemented, to be lowered in given hours, days or year periods, in which a relevant presence of route unfamiliar drivers, e.g. a noticeable touristic flow, is expected), and/or with policies aimed at achieving the homogeneity of driving behaviours.

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5. The regulatory framework

5.1 Italian standards for the safety management of the road network

5.1.1 Introduction

The purpose of this chapter is to illustrate the current regulatory framework for upgrading existing roads in Italy. This paragraph is based on "*Infrastrutture stradali più sicure- fase 2: le applicazioni*"[1], which was written with the contribution of P. Colonna, one the authors of this book.

In order to correctly identify the roads addressed by the content of this paragraph, it is necessary to make a due premise with respect to the terms used in this text. The term "rural roads" must not be strictly interpreted according to the provisions of Articles 3 and 4 of the Italian Road Regulations: roads that fall outside the perimeter of the built-up area (according to specific decision of the municipal council) but rather more generally, in relation to the territorial context crossed by the road. Therefore, this paragraph is also relevant for the road segments which, although falling within the perimeter of the built-up area, are functionally part of a rural network and which cross an environmental context like the rural one. On the other hand, this paragraph is not intended for the urban road network, which has clearly different characteristics and peculiarities.

The regulative references that will be taken as reference herein for the Italian roads are the following:

- Article 13 of Legislative Decree n. 285 of 30 April 1992: *New Italian Road Regulations* (henceforth briefly referred to as D.Lgs. n. 285/1992).
- Ministerial Decree n. 6792 of 5 November 2001: *Functional and Geometric Road Standards – Norme Funzionali e Geometriche delle Strade* (henceforth briefly referred to as D.M. n. 6792/2001*)*.
- Ministerial Decree n. 67/S of 22 April 2004: *Amendment of the Decree of 5 November 2001 – Modifica del decreto 5 novembre 2001* (henceforth briefly referred to as D.M. n. 67S/2004).
- Ministerial Decree of 19 April 2006: *Functional and Geometric Rules for the Construction of the Road Intersections – Norme Funzionali e Geometriche per la Costruzione di Intersezioni Stradali* (henceforth briefly referred to as D.M. 2006).
- Legislative Decree 15 March 2011 n. 35: *Actuation of the European directive 2008/96/CE about the road infrastructure management – Attuazione della direttiva 2008/96/CE sulla gestione della sicurezza delle infrastrutture stradali* (henceforth briefly referred to as D.Lgs. n. 35/2011).
- Ministerial Decree n. 137 of 2 May 2012: *Guidelines for the management of road infrastructure safety pursuant to art. 8 of Legislative Decree no. 35 of 15 March 2011 –Linee guida per la gestione della sicurezza delle infrastrutture stradali ai sensi dell'art. 8 del decreto legislativo 15 marzo 2011, n. 35* (henceforth briefly referred to as D.M. n 137/2012).

The act of regulation for road constructions, which has its origins in article 13 of the D.Lgs. n. 285/1992, is the D.M. n. 6792/2001. These regulations, initially conceived both for the construction of new road segments and for the upgrading of existing road segments, were limited by the subsequent D.M. n. 67S/2004, to be only applicable to new roads and used as a reference for the adjustment of existing roads. In addition, these standards provide both the issuing of specific rules for the upgrading of existing roads and the preparation of specific guidelines containing the criteria and methods for the applications for exemptions mentioned in the paragraph 2 of art. 13 of the D.Lgs. n. 285/1992, in accordance with the procedure laid down in Article 3 of the D.M. n. 6792/2001. It is to be considered that this amendment may have arisen from the difficulty encountered in the application of the rules annexed to the D.M. n. 6792/2001 for the adjustment of existing roads in an extremely complex territory from both an orographically and environmentally/anthropic point of view, such as Italy.

Although the D.M. n. 67S/2004 stated the specific rules for the adjustment of existing roads and guidelines for derogation requests to be issued within six months, no such Decree has in fact been issued up to date.

[1] Comitato Tecnico 3.2 AIPCR Progettazione, Gestione ed Esercizio di Infrastrutture Stradali più Sicure (2014), *Infrastrutture Stradali più Sicure Fase 2: Le Applicazioni*, presentato al Convegno Nazionale AIPCR 2014, Roma.

Therefore, in this situation, the provisions of art. 4 of the D.M. n. 67S/2004 continue to be applicable as a transitory regulation. The adjustment designs for existing roads must contain a specific report in which the aspects connected with safety requirements are analysed and in which it is demonstrated that the interventions can produce not only a functional improvement in traffic but also a safety improvement of the infrastructure. This provision, has highlighted, ten years after its issuing, its inadequacy due essentially to the absence of rules that would have been useful to provide the required demonstrations, and to the body in charge to assess the adequacy of such demonstrations.

However, it should be remembered that in March 2006 a draft of *Standards for the adjustments on existing roads" - Norme per gli interventi di adeguamento dell estrade esistenti* (Infrastructures and Transport Ministry - General Inspectorate for Traffic and Road Safety – March, 21st, 2006) has actually been prepared by a Commission set up for this purpose. Nevertheless, this Commission has never obtained the expected opinion from the Superior Council of Public Works. The same Superior Council for Public Works has also expressed its opinion on this draft: it says that in absence of the formal enactment of the D.M. n. 67S/2004, the relative draft issued in March, 2006, can only take the value of technical literature. Therefore, according to current legislation, the regulatory references for road upgrading interventions remain both the D.M. n. 6792/2001 and the Art. 4 of the D.M. n. 67S/2004.

Regarding Road Safety, the D.Lgs. n. 35/2011 *Actuation of the European directive 2008/96/CE about the road infrastructure management – Attuazione della direttiva 2008/96/CE sulla gestione della sicurezza delle infrastrutture stradali* has been issued. The main objective of the D.Lgs. n. 35/2011 is introducing interventions and procedures that improve safety of road infrastructures. In accordance with the Directives of the European Union, the D.Lgs. n. 35/2011 provided for such procedures to be applied immediately on the trans-European road network, and then gradually extended to all the other roads within a deadline stated by the art. 1.

In accordance with article 8 of the same D.Lgs. n. 35/2011, the *Guidelines for road infrastructure safety management – Linee Guida per la gestione della sicurezza delle infrastrutture stradali* were issued with the D.M. n 137/2012. The Guidelines are suitable to facilitate the application of the procedures relating to checks on road infrastructure designs (also adjustments) and safety provisions on existing roads. In addition, these guidelines rationalized all the activities related to the safety of road infrastructures through the organization of a cycle of activities that allows a complete management. The new regulatory body, which is introduced by the D.Lgs. n. 35/2011, does not modify in any way the provisions of the previous D.M. n. 67S/2004, which therefore maintains its full validity. It follows that designs for upgrading roads, for which it is necessary to carry out the safety checks provided for by art. 4 of the D.M. n. 67S/2004 must contain a specific report. Attention is drawn to the definitions contained in the letters d) and i) of article 2 of the mentioned Legislative Decree. This article specifies in point d) that "road safety control" is for infrastructure and for adjustment designs that involve changes to road layout. In the following point i), it defines "infrastructure design" as the design relating to the construction of a new road infrastructure, or a substantial modification of existing road infrastructure, with effects on traffic flows.

The guidelines issued with the D.M. n 137/2012 indicate interventions on existing roads to which "road safety control" is applied. In conclusion, from a coordinated reading of these definitions, it follows that safety checks, provided by the D.Lgs. n. 35/2011, it is not binding for an adjustment design of existing roads.

Basically, in adjustment designs of existing roads, the definition of the right balance between full compliance with the provisions of the D.M. n. 6792/2001 and the adoption of different technical solutions (for example reuse of existing structures and plants, presence of environmental or anthropogenic obstacles that cannot be easily eliminated, economic disproportion between the expected benefits and the necessary costs, time extensions due to the required procedures, unavailable budget, etc. ..) is jointly the task of:

- the designers: they must draw up the report required by art. 4 of the D.M. n. 67S/2004, in line with the provisions introduced by D.Lgs. n. 35/2011;
- the person in charge of verifying the design, in line with art. 112 of Legislative Decree 163 of 2006: *Code of public contracts for works, services and supplies implementing Directives 2004/17/EC and 2004/18/EC - Codice dei contratti pubblici relativi a lavori, servizi e forniture in attuazione delle direttive 2004/17/CE e 2004/18/CE*;
- the commissioning body, responsible for approving the project.

However, it is evident that the transitional regime, defined by art. 4 of the D.M. n. 67S/2004 is affected by technical limits. Indeed, no useful rules are provided to the designer for the report drafting and, at the same time, no useful tools are provided to the contracting authority for the enhancement of an objective evaluation. This regime is also affected by limitations attributable to the absence of a systemic view of the question under consideration. The report referred to in the article of the decree, having a general character and therefore being able to use also qualitative judgements, assumes an indicative value for the purposes of justifying the design choices, adopted for each individual intervention. Indeed, there are no requirements for which this report should also contain an analysis of the adjustment intervention impacts on the overall road network in which it is located.

On the contrary, the analysis of the individual adjustment intervention within the reference road network (already broadly introduced in the draft of rules for the adaptation interventions on existing roads of March 2006) appears to be an essential element also according to the recent D.Lgs. n. 35/2011. Such analysis is a fundamental requirement with respect to the planning logic.

Therefore, the report referred to in article 4 of the D.M. n. 67S/2004, which must demonstrate that the intervention on the existing road is capable of producing an increase in the level of safety, should be drawn up using rational criteria, based on the most accredited scientific knowledge in the field.

The following paragraphs will deal with the possible preparatory activities to be carried out, and in particular the basic data to be acquired to allow an exhaustive examination of the road conditions on which action is required, the relationships that can occur between the possible technical solutions to be adopted, the road conditions and the phenomenon of road crashes. This description is made with the aim of providing both designers and contracting stations a useful contribution for the choice of the most appropriate technical solutions to be adopted in the project of an adjustment intervention. The interventions must be selected according to the specific road characteristics, in order to increase the road safety, as expressly required by the ministerial D.M. n. 67S/2004, in relation to the available resource despite the budget constraints.

Intersections are a major problem for road safety and therefore they deserve specific attention. In correspondence of the intersections, different traffic flows converge. Therefore, their geometric characteristics are strongly correlated to traffic safety. Consequently, the link between the intersection characteristics and crashes has been studied extensively.

Here, however, given that the specific theme is the road adaptation sections and not the existing intersections, these links will not be deepened. The intersection is therefore treated only as a singular element which must be subjected to the intervention of adjustment. From this point of view, the designer must carefully consider how changes to the layout can affect the driver's approach to the intersection, in terms of approaching speed and also in terms of driver's workload and caution level. In addition, road adjustment interventions can change the way users approach junctions: sight distance, available for all traffic flows, different steering angle manoeuvres, size of the vehicles etc. According to these assessments, it will therefore be necessary to adapt the geometry and other intersection characteristics to make them congruent with the changed road conditions on which it is located.

The introduction of each new singular element on an existing road must always be provided to the driver. For example, in case of an intersection converted into a new roundabout, it is necessary to choose an adequate position and provide a reorganization of carriageway in the approaching sections and the creation of traffic islands of adequate length. In the event that the road under adjustment process albeit belonging to an urban network, functionally belongs to the suburban network and it has a section with double-carriageway roads, it must be underlined that the D.M. 2006 does not allow the construction of a roundabout. This is different to what is expressed by D.Lgs. n. 285/1992 which instead provides the possibility of traffic-lighted roundabouts.

With regard to the regulations applicable to the adjustment of existing intersections, it should be noted only that the D.M. 2006 constitutes "*the reference to which the design must aim*".

However, unlike the provisions of the D.M. n. 67S/2004 for road segments, no specific report is required in the event that the adjustment design of the intersection does not fully comply with the standards annexed to the D.M. 2006. Whether it is not possible a full compliance with the standards annexed to the D.M. 2006, the most appropriate solutions to be adopted to improve the safety conditions at the intersection must be identified with a safety analysis.

The main contents of the D.Lgs. n. 35/2011, which transposed Directive 2008/96/EC and came into force in Italy on 23 April 2011, are explained below. Note that, on October, 23rd 2019 the European Directive 2008/96/CE *Road infrastructure safety management* was updated (2008/96/CE). However, as the Italian national legislation has not yet been amended, it was decided to show what is currently required by the Directive 2008/96/CE.

The text refers to some concepts relating to Road Safety, which will be further specified in the chapter related to the Highway Safety Manual Method, in which the readers will find more details about definitions and in-depth analyses.

5.2 Guidelines criteria and modalities of road safety checks on designs, of safety inspections on existing infrastructures and of the implementation of the road network safety classification process[2]

The guidelines (D.M. n 137/2012) are divided in four parts:

1. General part
2. Road infrastructure safety management
3. Road safety checks on designs
4. Road infrastructure safety inspections.

5.2.1 General part

5.2.1.1 Object and purpose

The aim of guidelines for road infrastructure safety management are:

- to establish the criteria and modalities for design controls and safety inspections of existing infrastructures;
- to orient and to coordinate the activities of all people involved (territorial authorities, competent bodies, road owners and managers, design controllers, existing road inspectors);
- to give addresses for the procedures for road safety analyses and for the classification of the road network.

5.2.1.2 Regulatory framework

- The Community Directive 2008/96/EC on road infrastructure safety management aims at improving the level of safety of roads belonging to the trans-European road network (TEN).
- D.Lgs. n. 35/2011 implemented Directive 2008/96/EC and came into force on 23 April 2011.
- Article 12, paragraph 5 of D.Lgs. n. 35/2011 stated that, prior to the adoption of the Guidelines, in the transitional phase, the "Guidelines for road safety analysis" (Circular of the Ministry of Public Works no. 3699 of 8 June 2001) constituted the reference standard for road safety analysis.
- The content of the guidelines enters into force on the day following their publication in the Official Journal (08/09/2012).

5.2.1.3 Scope of application

Road networks

The scope of application of Legislative Decree n. 35 of 2011 concerns roads falling within the Trans-European Networks (TEN); the Decree will then be progressively extended to all road networks falling within the national territory in both urban and suburban areas, but there are still extensions to the applicability of this Decree on all the national roads

From 2016, in fact, the Decree was intended to be applied to all the roads of national interest (ANAS roads), but due to the extension of terms, the new starting date has been set on the 1st of January 2021 (up to the current date). Moreover, by 31 December 2020, the regions and autonomous provinces will determine the application of the directive to infrastructure falling within the competence of the regions and local authorities.

In order to verify which activities are necessary, reference should be made to the “actual type of infrastructure”:

[2]All figures and tables in this chapter are translated or adapted from the Italian Ministerial Decree n. 137 of May, 2nd, 2012 (Attachment): *Guidelines for the management of road infrastructures safety*, pursuant to art. 8 of the Legislative Decree n. 35 of March, 15th, 2011 – *Linee guida per la gestione della sicurezza delle infrastrutture stradali* (Decreto Ministeriale n. 137, 2 Maggio 2012), ai sensi dell’art. 8 del Decreto Legislativo 15 Marzo 2011, n. 35.

- existing or complete, constituted by roads belonging to the primary national network, i.e. almost the entire motorway network and other international layouts;
- planned or programmed, constituted by new road designs;
- to be upgraded, designs for the adaptation of existing infrastructures.

Tab. 5.1: Activities required according to the D.Lgs. n. 35/2011, based on network type.

Type of Network		*Activities required under D.Lgs. n.35/2011*				
					Actions	
Current TEN Network (Dec.884/04/CE)	*Planned TEN Network (Ce 19/10/11)*	*Actual Type of Infrastructure*	*Network Analysis*	*Inspections*	*Regular Maintenance*	*Exceptional Maintenance + New Projects (VISS-Projects-Controls)*
Existing	Completed	Existing				unnecessary
Existing	To be upgraded	Existing but to be enhanced				
Designed	To be upgraded	Existing but to be enhanced				
Designed	Planned	Existing but to be adjusted				
Designed	Planned	New	unnecessary	unnecessary	unnecessary	

Type of design

- Designs relating to the new road construction infrastructure.
- Designs that produce a substantial modification of existing road infrastructure with effects on traffic flows (the analysis is not limited to the infrastructure covered by the intervention, but it extends to infrastructures whose traffic flows are affected by the implementation of the intervention).
- Adjustment designs involving changes on the layout.

Checks must be carried out for each design level (preliminary, final and blueprint), as well as for the construction and pre-opening phases and for the first operation year.

The transitional phase

- For the feasibility study phase, VISS (“Valutazione di Impatto sulla Sicurezza Stradale” – “Road Safety Impact Assessment”) and all checks on the various design levels are required for all infrastructures.

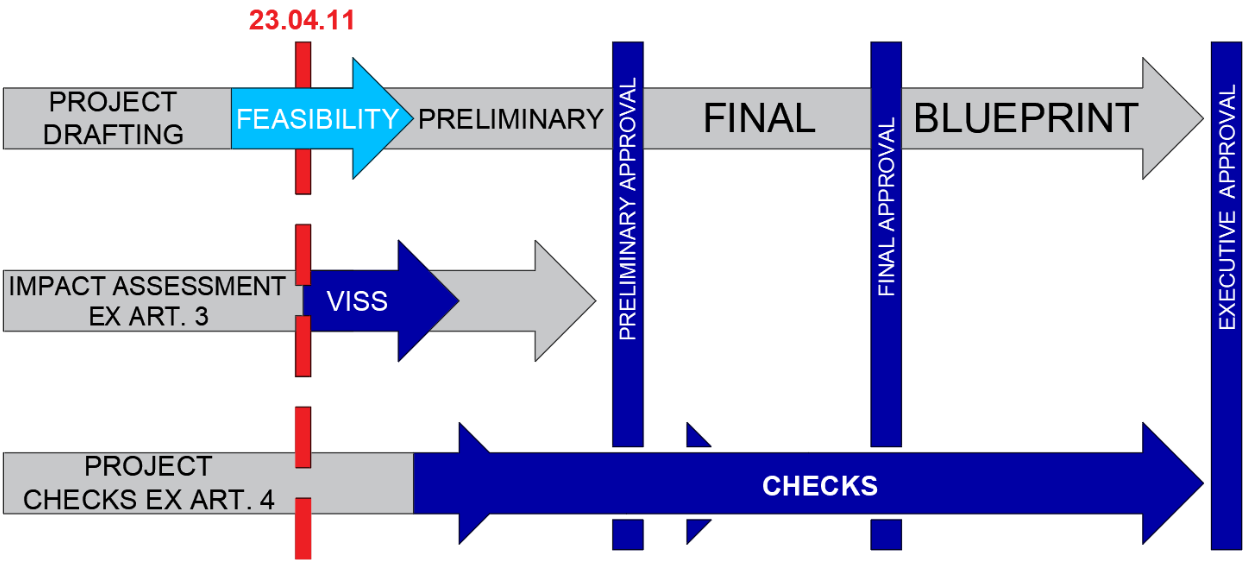

Fig. 5.1: Activities required for designs in the feasibility study phase.

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- For designs in the preliminary design phase, all checks on the various design levels are required for all the infrastructures in addition to the VISS.

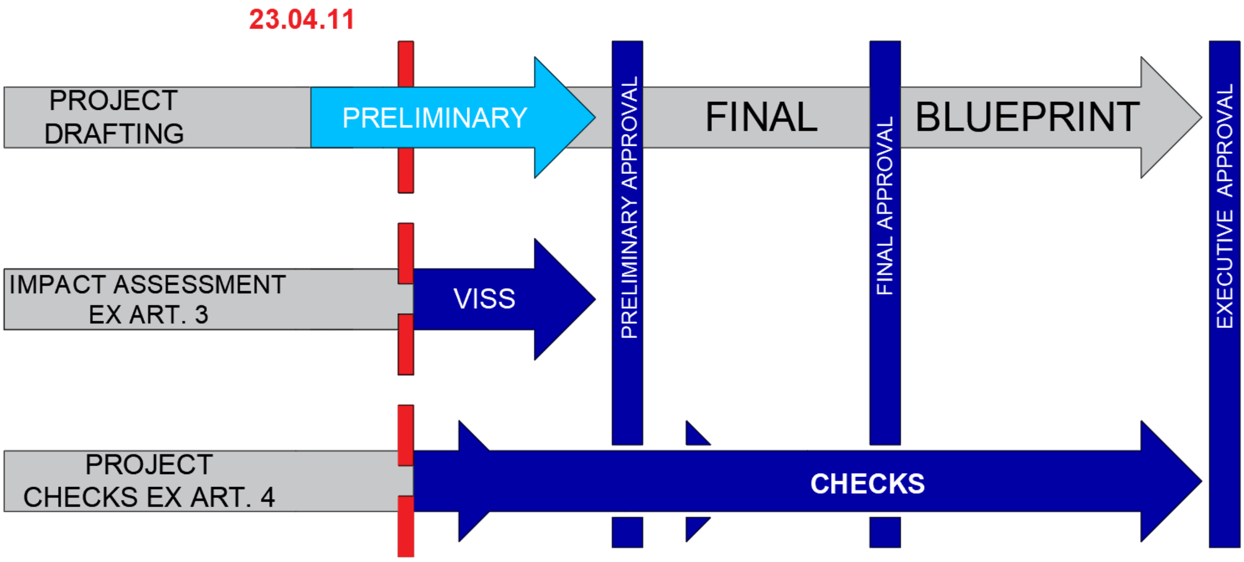

Fig. 5.2: Activities required for designs in the preliminary design phase.

- For designs in the final design phase, checks on both the final design and the blueprint are required only for ordinary infrastructures (excluding strategical infrastructures).

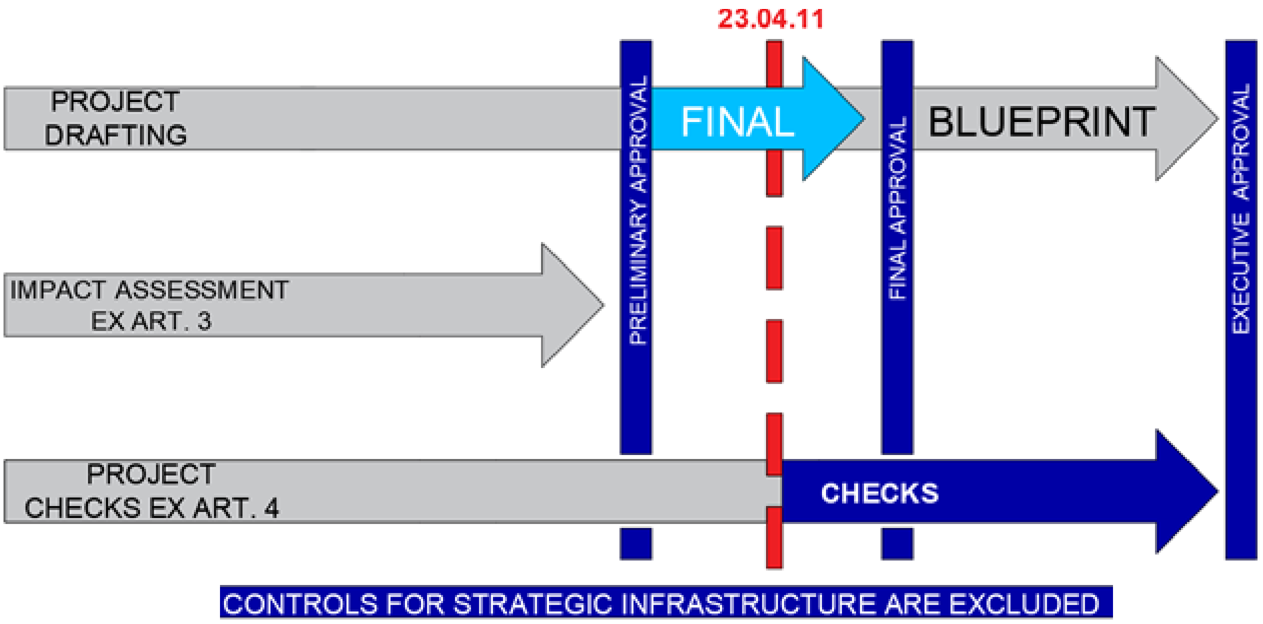

Fig. 5.3: Activities required for design projects in the final design phase.

- For all the infrastructures with the approved final design, i.e. at the blueprint design phase, checks are excluded.

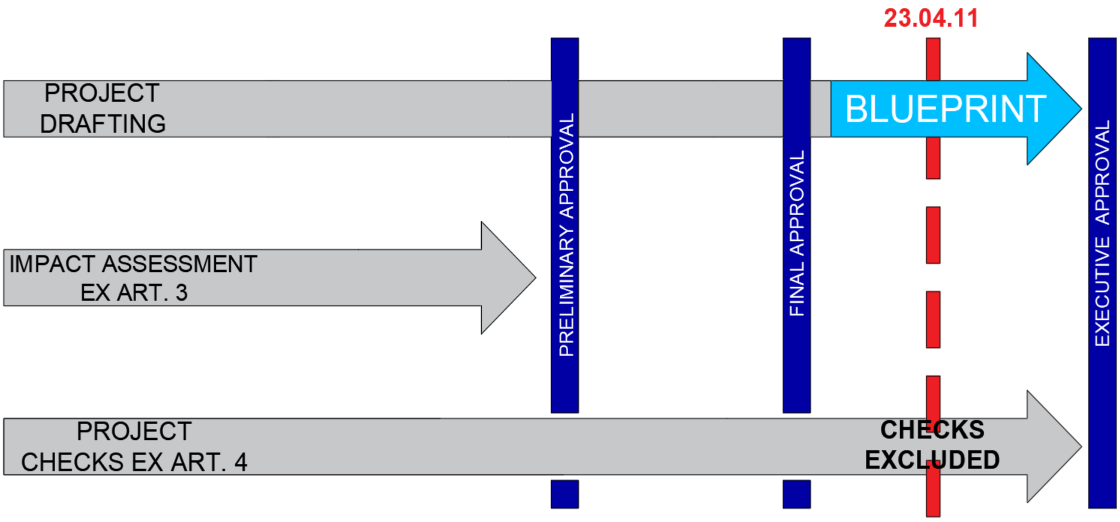

Fig. 5.4: Activities required for design projects in the blueprint design phase.

5.2.1.4 The subjects involved, and the functions performed

The purpose of the Guidelines (D.M. n 137/2012) is to address, coordinate and homogenise the activities of all people involved in the road infrastructure safety process:

- The *local authorities* responsible of the system regulations (State, Regions and Autonomous Provinces);
- the *competent bodies* (OC);
- *owners and managers* of roads;
- road safety *experts*, design *controllers* and existing roads *inspectors*.

The State

All the following competences have been assigned to the Ministry of Infrastructures and Transport:

- responsibility for the issuing of the decree implementation of the D.Lgs. n. 35/2011, valid throughout the national territory and for all road types, concerning the Guidelines, the Road Safety Impact Assessment (VISS), the training course programs for road safety experts and its updating, the fees for checks and inspections, the contribution of participation in training courses;
- responsibility for the management of road safety expert list, provisional and fully operational;
- the coordination of the permanent discussion table;
- determining the average social cost of a fatal crash and a severe crash;
- determining the total cost of the crash.

The Regions and Autonomous Provinces

They will have to regulate roads within their territory but not included in the TEN and the network of national interest:

- area of application, seen as the networks and types of roads on which the provisions of the D.Lgs. n. 35/2011 and the related implementing decrees are to be applied, even gradually over time;
- the modalities and timing of the implementation of the provisions dictated by them;
- the criteria and methods for identifying the relevant competent bodies.

The Competent Body

It performs the following functions:

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- classifies the road network of competence;
- responsibility for road inspections;
- responsibility for the tests performed on all the designs within the applicability area;
- planning and programming corrective interventions.

Owning agencies and Road managers

They perform the following functions:
- planning and programming new infrastructures or adjustment of existing infrastructures;
- management of the existing network they are responsible for.

Road safety experts

Road Safety Experts entrusted with design controls and inspections on existing roads must meet certain requirements: they must be adequately trained and have passed the professional certification examination.

5.2.2 Road infrastructure safety management

5.2.2.1 The complete cycle of planned activities

The road infrastructure safety management cycle consists of four macro activities.

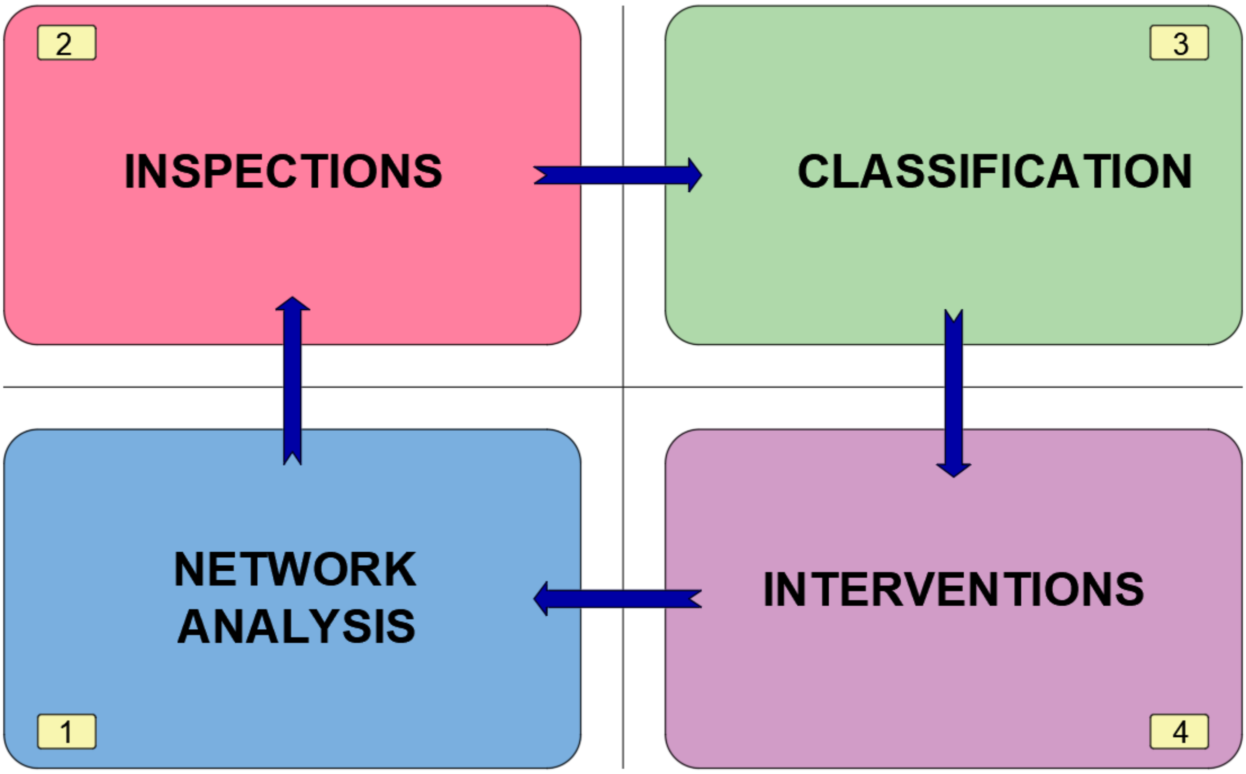

Fig. 5.5: Full cycle for planned activities

5.2.2.2 Analysis of the road network in the safety management

The first macro phase of the entire cycle is the analysis of the road network.

Each competent body carries out the examination of the road network operation for which it is responsible, in order to classify segments with high crash concentration (safety classification of the existing network).

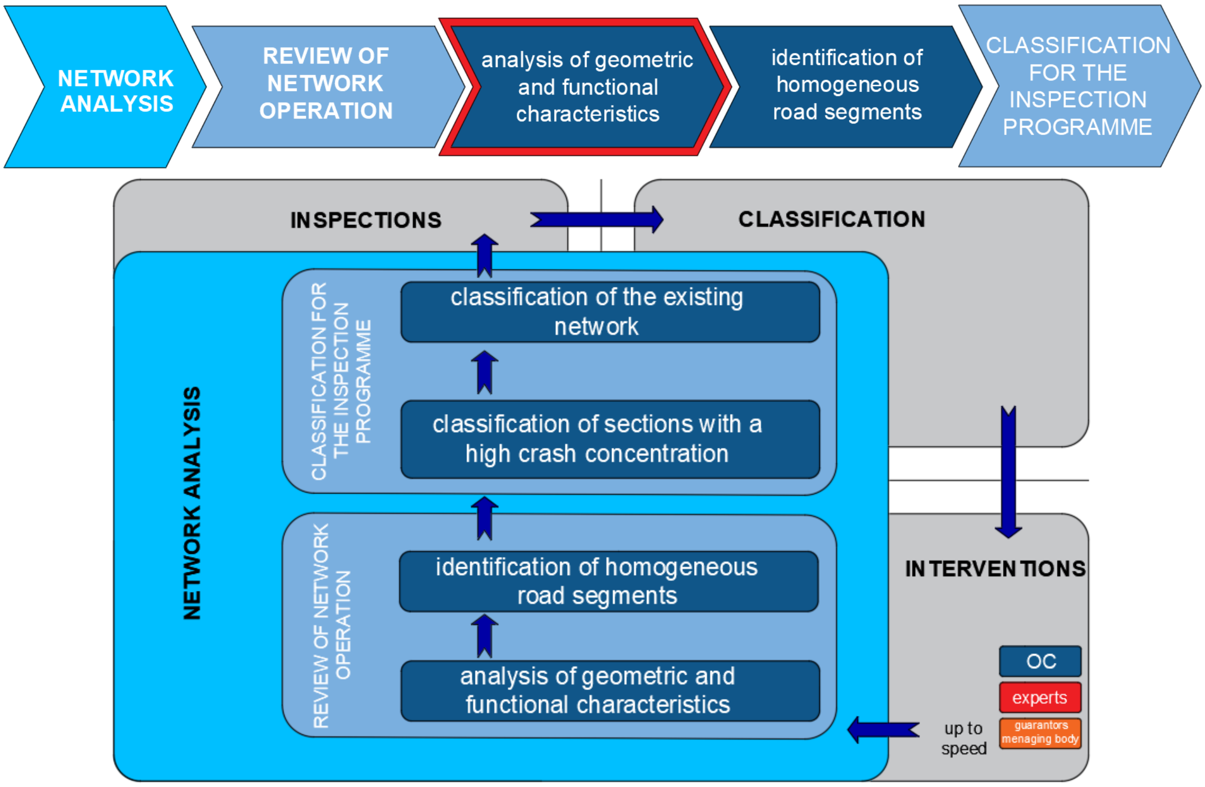

Fig. 5.6: First macro phase: road network analysis

The operation analysis of the road network must therefore be carried out by identifying the geometric and functional characteristics of elementary road segments which are homogeneous from a geometric and functional point of view.

In the urban context this identification requires even more attention in relation to the heterogeneity and fragmentation of the present infrastructural typologies.

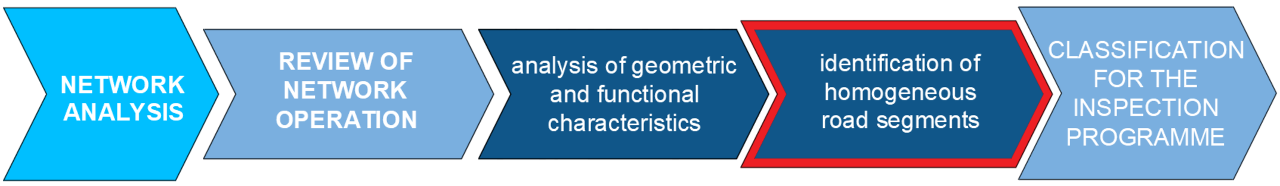

Fig. 5.7: Identification of homogeneous road segments

This identification is therefore based on the following elements:

- distinction between urban and rural areas;
- type of road (double or single carriageway);
- context and environmental integration (e.g. segment in flat or mountainous areas);
- functional class of the road (and possible further subdivisions within each class according to different cross-section displays, such as variation in the number of lanes);
- geometrical characteristics of the layout;
- traffic (volumes, components, density, time variability, etc.).

The road infrastructure is divided into "arcs", which are homogeneous road segments, and into "nodes". The latter are intersections and/or junctions or points where there is a discontinuity of geometric characteristics (road with variations, presence/absence of shoulders, etc.).

Fig. 5.8: Classification of critical segments

A list shall be drawn up of homogeneous road segments, classified according to crashes recorded in segments of the road network which are open to traffic for more than three years and where a considerable number of fatal crashes have occurred in proportion to the traffic flow.

The crash frequency is based on ISTAT (Italian National Institute of Statistics) data and, if available, on regional road safety data monitoring centres.

The crash event is due to the different component interactions in the system.

The user plays a central role in the system because he/she often represents, with his incorrect behaviour, the main cause of the crash event. The user correct perception of the geometrical and operational information of the road infrastructure is important in the crash phenomenon. Not only the user is a human, but also the infrastructure manager. Therefore, through the correct infrastructure management over time, humans ensure better functionality and greater safety.

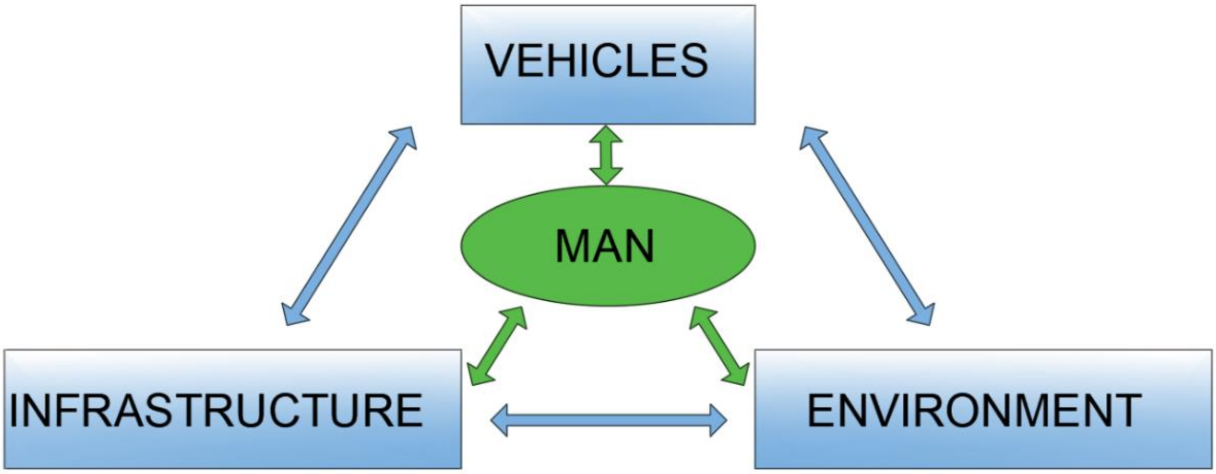

Fig. 5.9: System components.

The classifications of segments with a high concentration of crashes must be carried out by analysing fatal crashes as a priority (main objective of the D.Lgs. n. 35/2011). However, for the purposes of a more in-depth analysis of the crashes on the road network, it is appropriate to evaluate the same homogeneous segments also for "crashes with injuries".

The classification of segments with a high crash concentration represents the input for the classification of the network safety. It has the particular function, in first instance, of providing an indication to manage the priorities for interventions during an inspection schedule.

For the segments' classification with a high concentration of crashes, it is necessary to carry out an analysis of the crash data and use adequate crash metrics that can be adequately representative of the crashes as a whole.

The crash metrics must be calculated for each homogeneous segment and in proportion to the traffic flow, making a distinction by areas (urban and rural area) and by type of road (double and single carriageway).

The main data, necessary to be able to draw up the classification of the segments with a high crash concentration, are represented by:

- the length of the homogeneous road segment;
- the related crash data, recorded in the three-year period preceding the analysis and expressed as the annual average value of the number of deaths, injuries and crashes, and the average annual flow.

The following table schematically shows the main crash and traffic data necessary for the classification of segments with a high crash concentration, to which values are attributed by way of example:

Tab. 5.2: Main data needed for analysis.

Homogeneous segment	*Length*	*Deaths*	*Injuries*	*Crashes*	*Average annual flow*	*Total travelled km per year*
	km	*n.*	*n.*	*n.*	*10^6 vehicles*	*10^6 vehicles*km*
A	5	1	3	5	3	15
B	3	1	3	5	6	18
C	2	1	3	5	4	8

The crash analyses, aimed at the inspection program, are carried out according to the evaluation of annual data. For this reason, traffic parameters must also refer to the same period. In the case of rural freeways, the annual average daily traffic (AADT) can be used for example, as a basic parameter in order to have an estimate on an annual basis. In urban areas, further crash parameters may be used if the OC (competent body) deems them representative. The OC will have to classify segments with a high crash concentration in its network. The OC will have to get organised and oriented for subsequently updating the three-year classification of the existing network. The analyses based on the mere observation of crash numbers would lead to inaccurate results. These results generate intervention programs not very effective and with less benefits in terms of safety. The identification of crash metrics must therefore be carried out with adequate procedures which are capable to take into account the statistical variability of the phenomenon. According to the D.M. n 137/2012, the crash rate is the metrics to be privileged for the preparation of the segment classification with a high crash concentration. These metrics provide adequate information about the danger of each individual road segment according to the actual flow which travel on the road.

The main types of metrics, to be used for the classification of segments with a high crash concentration (also detailed in table 5.3) are listed below:

- crash rate (also expressed as a function of traffic flows);
- crash frequency (also expressed as a function of the kilometres);
- number of crashes.

The segment classification with a high crash concentration is defined in art. 2, c.1 letter e) of the D.Lgs. n. 35/2011 and it is based primarily on the assessment of homogeneous segments based on the significative number of detected fatal crashes. For this reason, these three metrics must refer primarily to the fatal and not to the global crash data. Table 5.3 shows, with a simple numerical example, the calculation of the three previous metrics for "fatal" and "injury" crashes, using the data of the previous table 5.2.

Tab. 5.3: Example 1 of crash classification according to traffic flows.

Metrics		*(a)*	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*	
Homogeneous segment	*Length*	*Death rate/ Vehicle flow*	*Death frequency*	*Deaths*	*Injury rate/ vehicle flow*	*Injury frequency*	*Injuries*	*Classification Indicator (a)*
	km	*N. deaths/10^6 veic.*km*	*N. deaths/ km*	*N.*	*N. deaths/ 10^6 veic.*km*	*N. injuries/ km*	*N.*	
A	5	1/15	1/5	1	3/15	3/5	3	2
B	3	1/18	1/3	1	3/18	3/3	3	3
C	2	1/8	1/2	1	3/8	3/2	3	1

The last column shows the classification according to the indicator (a). This classification shows that the homogeneous section C, whose length is 2 km, is the first one to have to be inspected because its mortality/vehicle flow rate (equal to 1/8) is the highest. In case traffic flows data are not available; the OC must carry out analysis based only on crash data. As a result, this analysis would be incomplete and would not fully meet the needs of classifications. Using metrics b) and d), the classification would be different:

Tab. 5.4: Example 1 of crash classification in case of unavailable traffic flows.

Metrics		*(a)*	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Homog. segment	*Length*	*Death rate/ Vehicle flow*	*Death frequency*	*Deaths*	*Injury rate/ vehicle flow*	*Injury frequency*	*Injuries*	*Classif. Indicator (b)*	*Classif. Indicator (e)*
	km	*N. deaths/10^6 veic.*km*	*N. deaths/ km*	*N.*	*N. deaths/ 10^6 veic.*km*	*N. injuries/ km*	*N.*		
A	5	Unavailable	1/5	1	Unavailable	3/5	3	3	3
B	3	Unavailable	1/3	1	Unavailable	3/3	3	2	2
C	2	Unavailable	1/2	1	Unavailable	3/2	3	1	1

Finally, a further and different numerical example is given in absence of traffic data and in case of use of different crash data from which a different ranking emerges, according to the different indicators chosen, such as indicator b) or indicator e):

Tab. 5.5: Example 2 of crash classification in case of unavailable traffic flows.

Metrics		*(a)*	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Homog. segment	*Length*	*Death rate/ Vehicle flow*	*Death frequency*	*Deaths*	*Injury rate/ vehicle flow*	*Injury frequency*	*Injuries*	*Classif. Indicator (b)*	*Classif. Indicator (e)*
	km	*N. deaths/10^6 veic.*km*	*N. deaths/ km*	*N.*	*N. deaths/ 10^6 veic.*km*	*N. injuries/ km*	*N.*		
A	5	Unavailable	7/5	7	Unavailable	3/5	3	2	2
B	3	Unavailable	1/3	1	Unavailable	2/3	2	3	1
C	2	Unavailable	6/2	6	Unavailable	1/2	1	1	3

Therefore, it is much more important to indicate the priority of the various metrics to be used for the segment classification with a high crash concentration. It is useful to proceed with classification even in the absence of some data necessary for the definition of the most significant metrics. However, the OC will have to try to acquire data in any case for the purposes of the subsequent three-year classification. Table 5.6 below indicates the priority of the various metrics to be used for the segment classification with a high concentration of crashes which must refer primarily to fatal ones (as required by the D.Lgs. n. 35/2011).

The next table shows how metrics belonging to the group with priority 1, give the most important information for the purpose of the road network crash classification. The number of deaths/injuries/crashes, recorded for each homogeneous road segment, is correlated with road segment length and with traffic flows. Metrics belonging to the group with priority 2, provide information on crash data related to the road length only. Finally, metrics belonging to the priority 3 group, provide only information on the number of crashes and number of deaths and injuries.

Tab. 5.6: Crash metrics to be used for classification.

Priority	*Crash metrics*	*Unit of measure*
1	Crash Rate with Deaths/Vehicle Flow	N. Crashes With Deaths/ Veic.*Km
	Crash Rate with Injuries/Vehicle Flow	N. Crashes With Injuries/ Veic.*Km
	Crash Rate/Vehicle Flow	N. Crashes /Veic.*Km
	Death Rate/Vehicle Flow	N. Deaths/ Veic.*Km
	Trauma Rate/Vehicle Flow	(N. Deaths + N. Injuries)/Veic.*Km
	Injury Rate/Vehicle Flow	N. Injuries/Veic.*Km
2	Death Crash Frequency	N. Crash Deaths/Km
	Injury Crash Frequency	N. Crash Injuries/Km
	Crash Frequency	N. Crashes/Km
	Death Frequency	N. Deaths/Km
	Injury Frequency	N. Injuries/Km
3	Death Rate	N. Deaths/N. Crashes
	Trauma Rate	(N. Deaths + N. Injuries)/N. Crashes
	Injury Rate	N. Injuries/N. Crashes
	N. Deaths	Number
	N. Injuries	Number
	N. Crashes	Number

These metrics must be considered complementary and not alternative because they are able to represent specific aspects of safety. The OC will have to evaluate the most appropriate metric (in the order they are listed in table 5.6) according to area, territorial context and type of road. The OC has the right to use metrics, referring to seasonal periods of time, in which there are high traffic flows and crash events. If metrics were reported on an annual basis, they could lead to incorrect assessments on the crash number.

In the absence of exhaustive and complete data for the crash metrics, indicated in table 5.6, the OC, in the initial phase of the provision implementation of the D.Lgs. n. 35/2011, may carry out other types of analysis (for example analysis of the different types of users, state of maintenance of the infrastructure, analysis of speeds, etc.). The network safety classification is conducted on the segments in the existing road network according to their potential for improving safety and saving costs related to crashes.

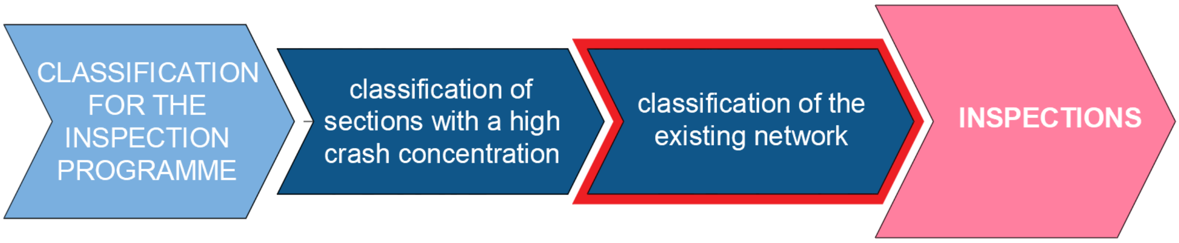

Fig. 5.10: The classification of existing network safety.

Using the Safety Potential-SAPO metric (D.M. n 137/2012), it is possible to assign an inspection priority to the various homogeneous segments in the network.

*SAPO = DCI (k€/km*year)* (Eq. 5-1)

where:
DCI = average density of crash cost = CAI / L;

- CAI (K€/year) = average annual cost of crashes = (Nm * Cm + Nfg * CfG + Nfl * Cfl);
  - Nm, NfG and NfL are number of deaths, severe and slightly injuries respectively;
  - Cm, CfG and CfL (K€) are the respective average costs of the dead, severely and slightly injured;
- L (km) = length of road segment;

BDCI = basic value of average crash cost density = (BTCI * 365 * ADT) / 10^6 (Eq. 5-2)

- BTCI (€ / 1000 * vehicle * km) = base rate of the cost of crashes;
- ADT (vehicles / day) = average daily traffic.

5.2.2.3 Inspection program

The second macro-phase of the entire cycle is made up of inspections.

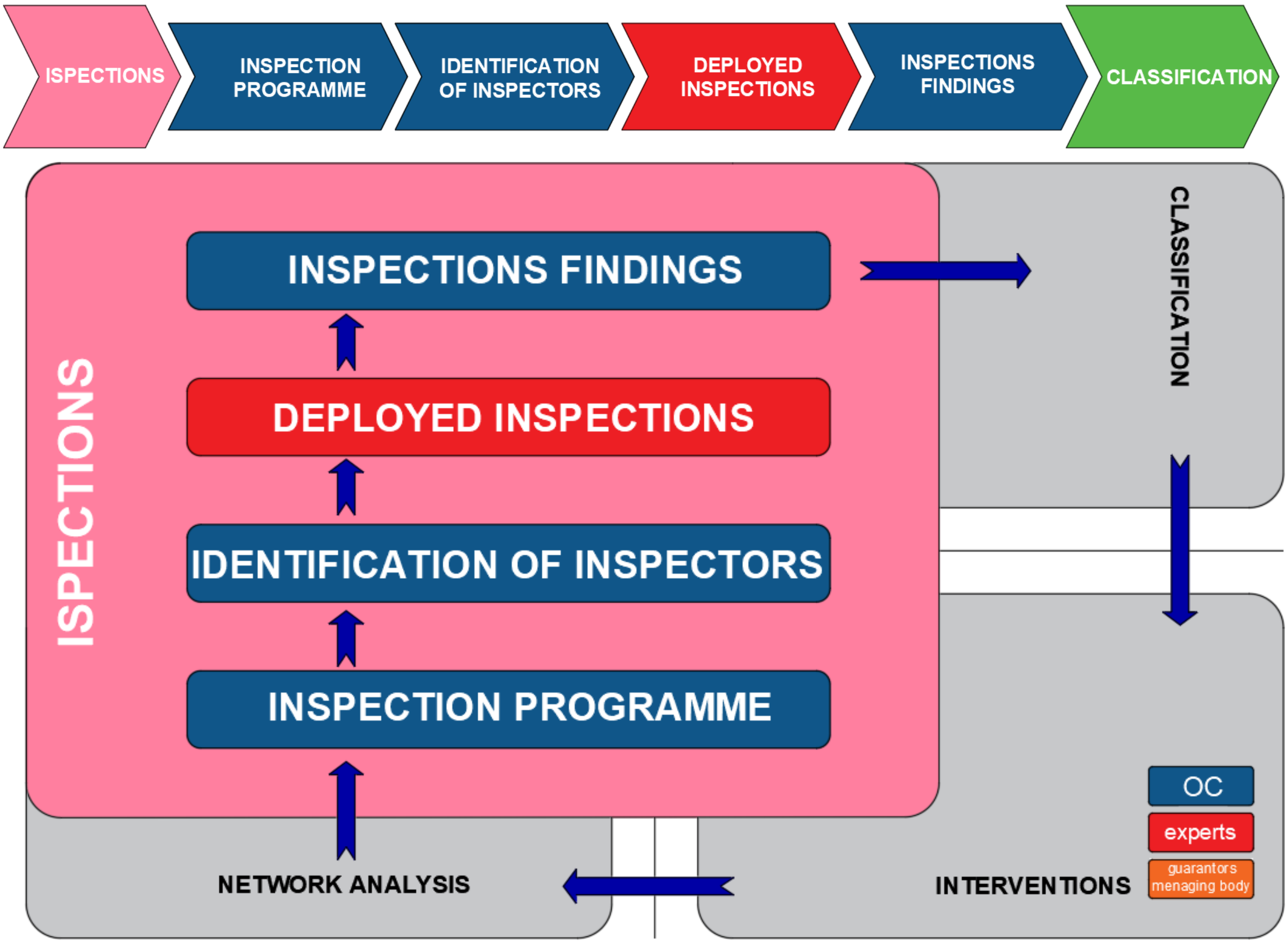

Fig. 5.11: Second macro-phase: inspections.

The phases of inspection activity are:

- *inspection program*: it is the responsibility of the OC; it allows inspections to be scheduled on road network according to efficiency criteria based on the safety potential of singular homogeneous segments;
- *identification of inspectors*: it is the responsibility of the OC; it allows an optimal use of available resources according to needs and priorities;
- *deployment of inspections*: it is the responsibility of safety experts; it is the central and most technical phase of the entire procedure, and it consists in the preparation of the final report structured in a series of recommendations, which are differentiated according to critical aspects of the infrastructure, and the consequent need for interventions;
- *inspection findings*: they are the responsibility of the OC and they are necessary for safety classification and for the consequent planning and programming of routine maintenance operations, which are immediately and extraordinarily implemented.

5.2.2.4 Classification for interventions' planning

The third macro-phase of the entire cycle consists of the classification process for intervention planning.

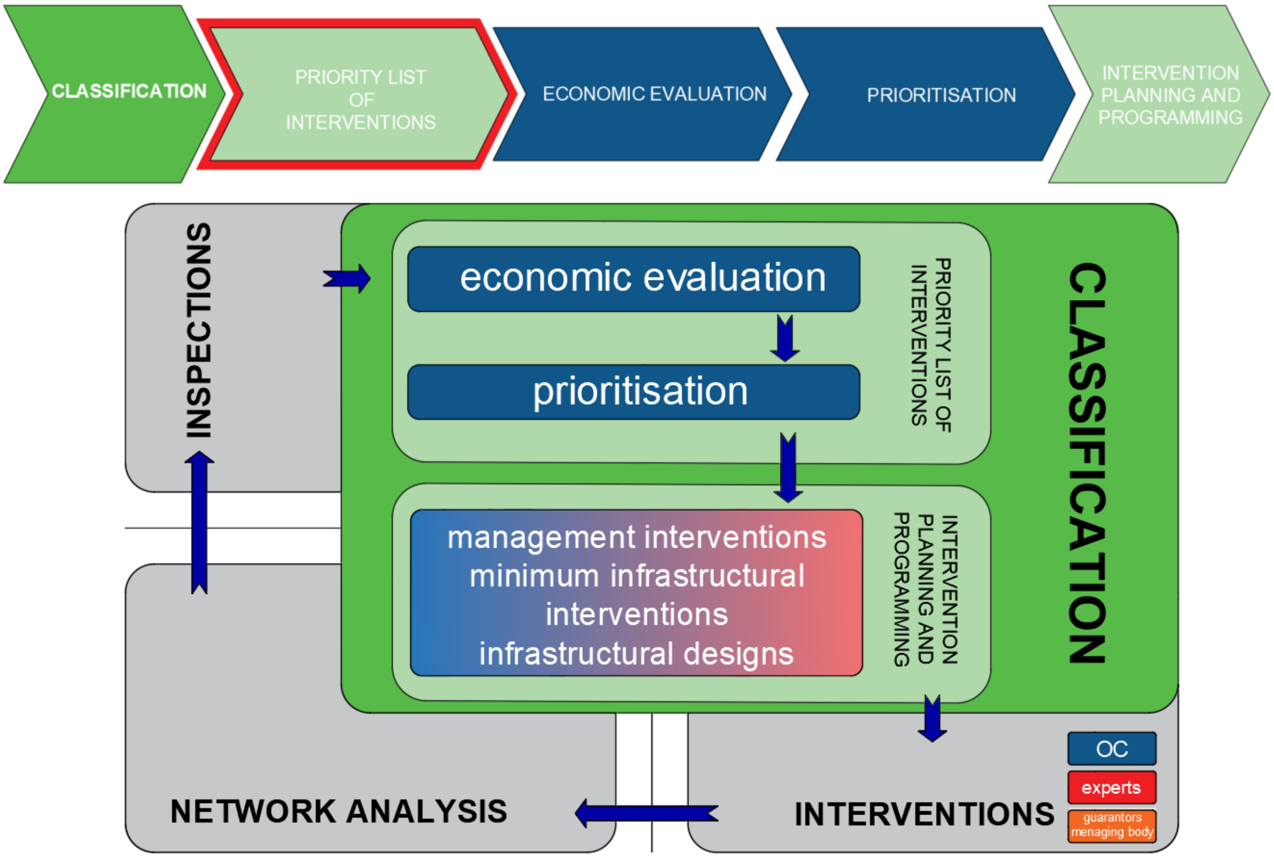

Fig. 5.12: Third macro-phase: the classification.

Once the accuracy and suitability of the proposals for corrective actions contained in the inspection results have been assessed, the OC distinguishes the proposals into two categories:

- *routine maintenance interventions*: necessary measures to solve maintenance deficiencies, the implementation of which is immediately requested from the managing body because they are highly effective and low-cost, and they are easily included in the ordinary management budget;
- *extraordinary maintenance interventions*: measures that require substantial changes to the infrastructure. For these measures, the OC, in order to identify the most suitable and affordable solution, proceeds to the economic intervention evaluation, which requires interaction with the owner/manager body of the road.

Fig. 5.13: Economic evaluation for programming purposes.

The principal method used to evaluate the technical and economic convenience of an intervention on the road network is the CBA (cost-benefit analysis).

The analysis takes into consideration all social benefits and costs which derive from the realization of the design. However, defining the monetary value of goods such as environmental or human life is not easy.

It is necessary to evaluate:

- *impacts on road safety:* the road crash costs (cost of human life, health costs, costs from lower production capacity and reduced quality of life, costs of material damages, administrative costs);
- *impacts on mobility:* travel time cost, congestion cost;
- *impacts on the environment:* noise and air pollution, visual intrusion, landscape impact.

Once the costs (C) and the benefits (B), located over the period of time (n) which represents the duration of economic investment, have been quantified and the discount rate has been defined, the judgment of convenience to carry out the intervention can be based on:

- *the net actual value (VAN):* discounted difference between benefits and costs, for a design without alternatives: convenience exists when the sum of the benefits is greater than or equal to the sum of costs. In the case of different design alternatives, the most convenient will be the one corresponding to the maximum VAN;
- *the discounted benefit/cost ratio (RBCA):* ratio between benefits and costs.

The intervention cannot therefore be considered convenient if the VAN is negative or if the discounted B/C ratio is less than 1.

Procedural scheme for the application of the CBA:

- identification of the purposes of the intervention and specific problem analysis for the case study;
- identification of the homogeneous road segment characteristics;
- network analysis affected by the intervention;
- crash data analysis (evolutionary trend in the last 3 years);
- description of the expected intervention;
- economic evaluation of the measure(s) to be adopted on the homogeneous road segment:
    - estimate of direct costs (implementation of the intervention and maintenance, etc.)
    - estimate of indirect costs (social costs from fatal and injury crashes, increase in air and noise pollution, etc.)
    - estimate of direct benefits (crash reduction, congestion reduction, etc.)
    - estimate of indirect benefits (travel time reduction, air and noise pollution reduction, etc.)
    - discounting of all analysed costs and benefits.
- possible joint analysis of several interventions relating to contiguous homogeneous road segments;
- calculation of the benefit-cost ratio.

Fig. 5.14: The identification of priorities for programming purposes.

The interventions must be oriented towards the *best benefit-cost ratio*:

- *minimal management and infrastructural interventions* are characterized by a low construction cost, immediate viability, and therefore with a high benefit-cost ratio that is more easily estimated;
- *"infrastructure designs"* are characterized by high costs and long construction times; they will produce their greatest benefits over a longer period.

Fig. 5.15: Intervention planning.

For the purpose of identifying the priorities of the interventions to be carried out, the OC must use an economic methodology (CBA analysis). This analysis must be kept homogeneous and coherent for the evaluations throughout the network of competence.

In addition, for the purposes of intervention planning and programming:

- ordinary maintenance interventions can be carried out immediately by the owner/manager. They do not require real time planning and scheduling, but their implementation must be effectively monitored and

controlled by the OC so that they can be carried out quickly, in the respect of the organization and internal planning of the managing body activities;
- for the "infrastructure designs", and all corrective interventions that require significant economic resources for their implementation, the OC requires to the owner/manager to plan the preparatory activities required by the D.Lgs. n. 35/2011 (VISS and design) as well as to find the necessary resources.

5.2.2.5 Implementation of interventions

The fourth macro-phase of the entire cycle consists of the implementation of interventions divided into ordinary and extraordinary maintenance:
- The *management interventions* are an exclusive responsibility of the managing body. It is subject to the supervision of the OC. For this reason, they may have modifications and/or additions both in quantitative and qualitative terms, as well as temporal advances (possibly delays in the event that they should be rendered consistent and compatible with interventions of other operators). Examples of such interventions are:
    - remaking and replacement of horizontal and vertical signs;
    - replacement of superficial-road-pavement-layer wear mat;
    - replacement of restraint devices;
    - plant maintenance (lighting, traffic lights, variable message signs, etc.)
- The *minimum infrastructural interventions* do not constitute "infrastructure designs" and do not involve changes to the layout. The OC approves the proposal of the managing body regarding implementation times and verifies their compatibility and consistency with other interventions. These interventions include:
    - isolated and insignificant layout adjustments;
    - realization of single accesses;
    - construction of a service area;
    - construction of parking area.

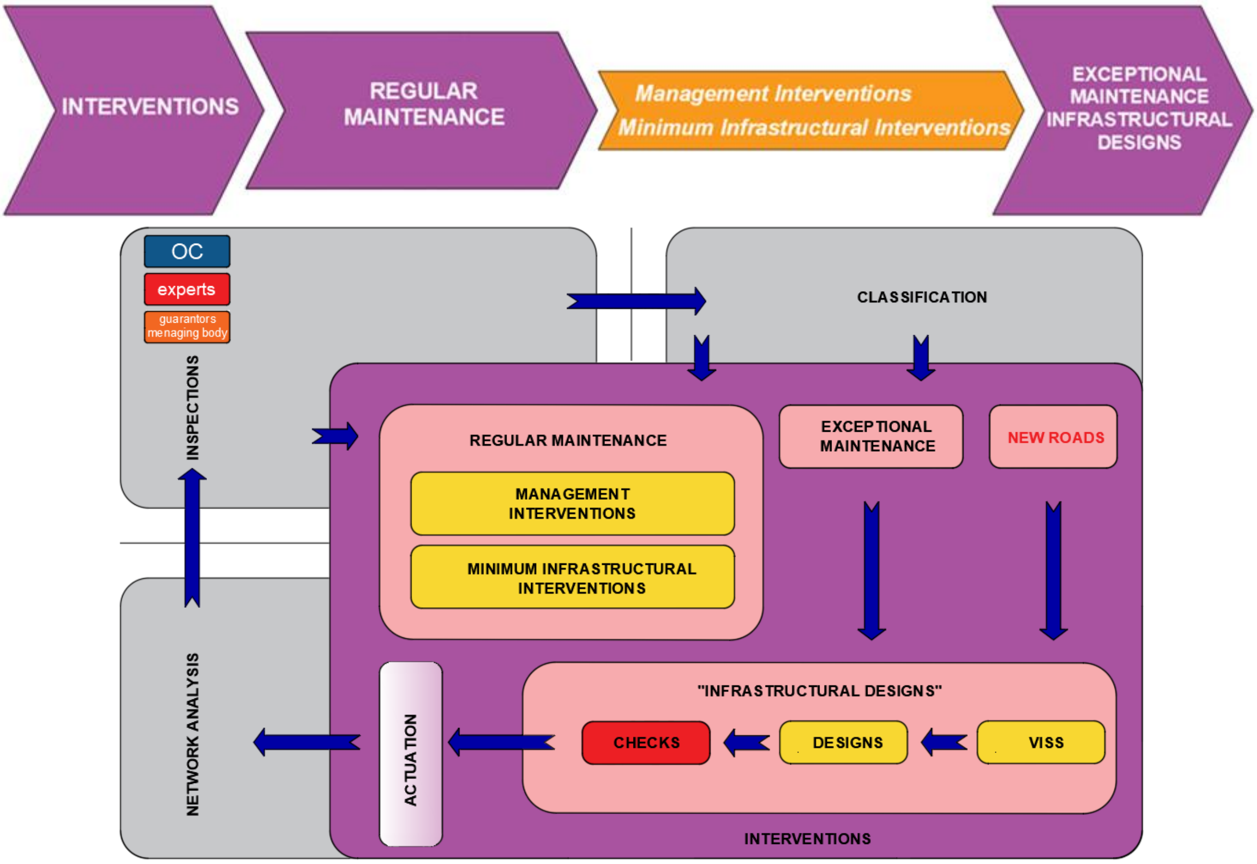

Fig. 5.16: Fourth macro-phase: the implementation of interventions.

Fig. 5.17: Activities of the second part of the macro-phase: Interventions.

The following procedure are applied on infrastructural interventions which involve layout changes:

- VISS (Road Safety Impact Assessment)

The VISS is a study conducted by the managing body during the planning phase and in any case prior to the approval of the preliminary design. The VISS is analysed by the OC in the context of feasibility or in the first level of design (art. 3 of D.Lgs. n. 35/2011).

- Designs

Designs must be produced, by the owner/manager, with all the design documents required by current regulations, and delivered to the OC, only those significant for the planned inspection.

- Checks

Therefore, the procedures stated for project controls apply to such interventions. These procedures involve a series of initial activities carried out on a documentary basis and then a series of inspection activities which are carried out during the intervention. The latter are the monitoring of interventions.

5.2.2.6 Monitoring of interventions

The further subsequent phase, represented by the monitoring of the interventions, allows the cycle to be closed. The identification of the effectiveness of the interventions is based on:

- crash metrics variation analysis before/after the intervention;
- traffic flows and traffic composition variation;
- analysis of detected speeds variation;
- application of *benefit-cost analysis (CBA)* with post-intervention data or evaluation of the intervention effectiveness through *cost-effectiveness analysis (CEA)*.

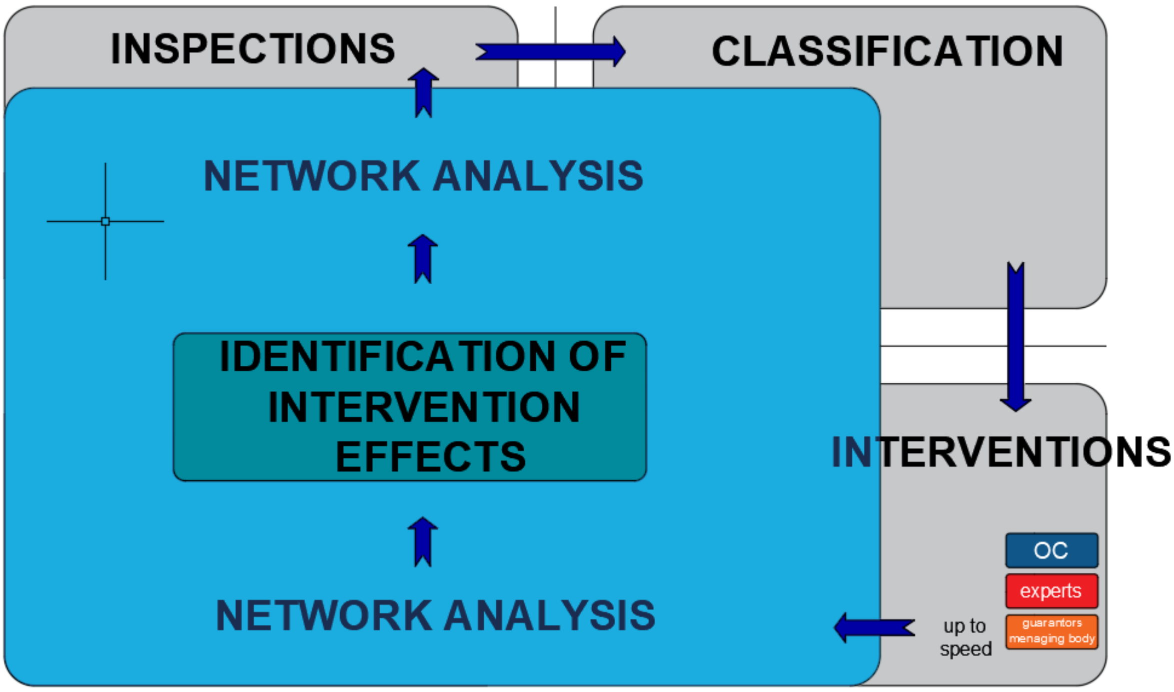

Fig. 5.18: Fifth macro-phase: the monitoring of interventions.

5.2.2.7 Cost-benefit analysis (CBA)

This analysis is carried out through the "before/after" method, for which performance measurements are carried out before and after the intervention implementation. The crash data refer to the period prior and after the implementation of the intervention.

The following aspects should be considered:

- *adequate observation period*: crashes can be considered casual events, and, for this reason, the process of evaluating results requires sufficiently long observation periods (more than only one year)
- *variability of the phenomenon and the influence of external factors*:
    - regression to the mean: crashes have a random trend over time, for which a frequency distribution can be assumed (e.g. Poisson). This means that the measured values may be different, in some periods, from the mean values that would have been expected in the same points;
    - crash migration: an event, verified in the period following the intervention, can cause a variation in the number of crashes due to completely random effects. These crashes could be incorrectly attributed to the intervention itself;
    - change in traffic volumes: it is a variability factor that can act on the crash numbers in the place where the intervention took place;
    - long-term trend: for some types of crashes, medium-long-term trend can be observed, determined by factors such as the improvement of vehicle equipment, a behavioural change of users, etc.;
    - coexistence of several interventions.

Empirical-Bayes" (EB) corrective models

The EB method allows to correlate the crash information data through the use of forecast models: Safety Performance Functions (SPFs).

The procedure is based on the joint use of the frequency and/or type of crashes expected on elements like the one analysed and of the safety performance function. The objective of the model is to determine the expected number (e.g. reduction of crashes) on the element in question.

Cost-Effectiveness Analysis (CEA)

CEA can be considered a simplified CBA where not all the effects can be monetized. The data necessary for the definition of the cost-effectiveness ratio are:

- *The number of avoided crashes* = the determination of the number of crashes avoided after the implementation of the intervention in the given time interval;
- *cost of the adopted measures* = cost of the measures necessary for carrying out the intervention;
- *cost per unit of result* = ratio between costs and effects of the intervention;
- *the result per unit of cost* = ratio between effects and costs of the intervention.

5.2.2.8 Return to the examination of the functioning of the network

All the interventions produce effects on the functioning of the network. For this reason, also the function of the network must be constantly examined because continuously evolving.

A continuous and constant update of the network analysis is essential for monitoring interventions and for increasing functionality and affordability. However, the OC must carry out the classification at least every three years.

In this way, the cycle of activities does not end but, due to this characteristic of continuity, it restarts for a new cycle aimed at a progressive improvement of the network.

5.2.3 Road safety checks on designs

5.2.3.1 Check purposes

The control of the design is preceded in the planning phase by the Road Safety Impact Assessment (VISS).

- The *VISS* is prepared by the owner and/or manager of the road. It constitutes a multi-criteria analysis that illustrates advantages and disadvantages of the various possible solutions. It assesses the planned intervention impacts in terms of safety of the infrastructure and of the entire network connected to it.
- The *check* is carried out on the only design solution already identified and it is performed by the competent body. Road safety checks on designs must be carried out for the preliminary, definitive and blueprint design and must also be carried out in the construction phase and in the pre-opening phase, in the first year of operation.

Purposes

The purposes are:

- to identify potential critical issues in the designs, as after construction, it could be extremely expensive or even unworkable to fix them;
- to ensure that the safety requirements, for all users, are considered at all stages of the road infrastructure design;
- to improve awareness of road safety aspects for all the bodies involved in the entire process.

Benefits

The benefits are:

- a general improvement in knowledge of road safety principles, with an improvement both in the design criteria and in the design rules;
- a reduced need to adapt road infrastructures after their construction;
- a lower infrastructure life cycle cost, resulting from the lower costs related to crashes.

The purposes of the checks can be outlined, in relation to:

- *type of infrastructure*: the analysis is not limited just to the network in which the infrastructure is included, but it must also be extended to other networks adjacent to it and with which it is interconnected.
- *design phase:*
    - *preliminary design phase:* to evaluate the basic choices and the basic settings of the design, the correspondence of the contents to the objectives pursued by the client and set by the designer, the effectiveness, efficiency and cost-effectiveness of the design solution, sufficiency or redundancy of interventions. (VISS and relevant design documents);
    - *definitive design phase:* to evaluate what emerged in the previous control phase of the preliminary design, the design solution accuracy from a technical-functional point of view with reference to the design standards and the reference standards, the identification of any critical safety aspects;
    - *blueprint design phase:* to evaluate the implementation of the indications contained in the final design control report, the design solution accuracy according to the degree of detail.
- *active phase of the project:*
    - *construction phase:* to assess the implementation of measures indicated in the check report of the blueprint design, to propose and to suggest to the Construction Manager, through the OC and the managing body, minimum design measures which are not predictable during the design phase;
    - *before opening to traffic:* to eliminate any critical safety aspects that emerged only at the end of the implementation of the intervention;
    - *first year of operation:* to verify the compliance of the effective functioning of the interventions according to the expected results;
- *area in which the design fits:*

- *rural area:* to analyse aspects such as the consistency of the technical-functional of road characteristics with those of the network to which it belongs to, the configuration of intersections and their inter-distance, the vertical-horizontal alignment coordination of the layout, the functional and street furniture components, the evaluation of the location of service areas, etc;
- *urban area:* to analyse the conformation of intersections, traffic components, traffic calming measures, safety conditions for vulnerable users, etc.

5.2.3.2 Types of designs to be checked

Checks must be carried out on:

- designs relating to the new built road infrastructures (always mandatory);
- designs that produce a substantial modification of an existing road infrastructure with effects on traffic;
- adjustment designs involving changes on the layout.

Regions and the autonomous provinces will regulate in detail the design scope to be checked, excluding very limited interventions or interventions falling under ordinary maintenance, as shown in the following example.

Tab. 5.7: Need for checks according to the type of intervention.

Necessary checks	*Unnecessary checks*
Infrastructure designs and interventions involving modifications of the layout	*Very limited infrastructural interventions, very limited management interventions*
Realization of alternative layout solutions	Temporary alternative route solutions
Coordinated series of layout rectification	Unique and not important correction of the route
Implementation and coordination of accesses	Implementation of a single access
Construction of service roads	-
Implementation of one or more lanes	-
Creation of a new intersection	-
Creation of a series of intersections	-
Conversion of an intersection (from standard intersection to roundabout or intersection to staggered levels etc.)	Little variation of intersection geometric characteristics

5.2.3.3 Stages of the control procedure

The beginning of the design project: manager's communication to the Competent Body

The managing body must inform the OC that the project has started. This step enables the starting of the road safety monitoring in all its phases, within an appropriate timeframe and without causing delays in the design process. The managing body therefore communicates to the OC all the elements related to the design assignment in terms of time and deadlines, also for all projects carried out internally without external assignment so that the OC can assess the need for control. In this way, the OC can assess the need for checks. After the communication of the formalization of the design assignment by the managing body to the OC, the OC, if the project falls within the cases provided by art. 4, c.1 of D.Lgs. n. 35/2011, must identify the controller and assign the task to him according to the established methods and timescales.

Identification of controllers

The following concerns the identification of controllers.

- Times

The OC, immediately after having received the communication by the managing body, starts the procedure for the identification of the controller. The OC should ensure that checks run parallel to the design.

- Modality

For each design, the OC identifies a single controller or multiple controllers according to the complexity of the design. A person who takes part or has already taken part directly or indirectly to the design draft, during the work directions or the design testing phase, cannot be commissioned. In the absence or unavailability of internal professionals, the OC will entrust the control of the designs to "personnel not belonging to the Competent Body".

Assignment of control

The following concerns the assignment of control.

- Checking topics

The control may concern a set of individual designs, if they are closely related. It is also appropriate to entrust together the control of the blueprint design, the control of the construction phase and the pre-opening to traffic, because they are closely related to the blueprint design. In the case of a project involving TEN tunnels longer than 500 m, the OC, in first instance, must request the opinion of the Standing Committee on Tunnels, as well as the CIG (Inter-Governative Commission) for cross-border tunnels, pursuant to Legislative Decree 5 October 2006 n. 264: Implementation of Directive 2004/54/EC on safety in tunnels in the Trans-European Road Network - Attuazione della direttiva 2004/54/CE in materia di sicurezza per le gallerie della rete stradale transeuropea.

- Responsibility of the controller

The control activity consists of check reports that must be reviewed by the designer. The managing body must justify to the OC the reasons for non-adjustment if the design is not adequate as written in the control report. It is then the responsibility of the OC to accept the justifications given by the operator or to adjust the design accordingly to controller's recommendations.

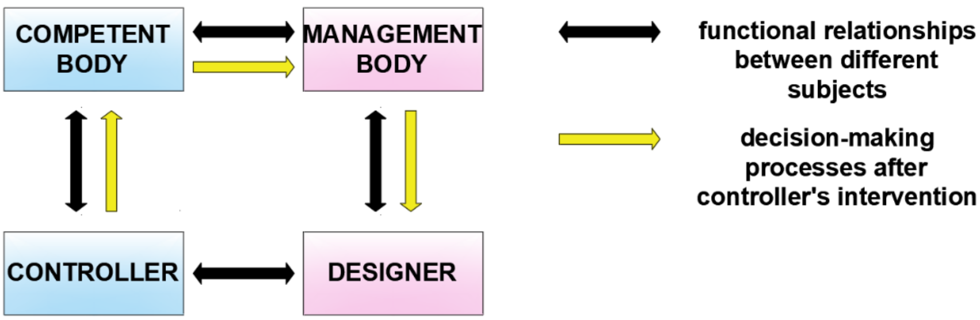

Fig. 5.19: Functional relationships between subjects.

The OC, after having received and shared the instructions by the controller and taken over responsibilities, imposes the same instructions to the managing body. The managing body then transfers them to the designer for the necessary adjustment of the project.

Check deployment

The following concerns the actions to check deployment.

- *Times*: for approval purposes, the checks must take place within a pre-established maximum time (starting from the delivery by the designer). The checks of the construction phase and of the pre-opening to traffic, must be programmed in order not to delay the construction or the opening to traffic. The control reports are checked and delivered by the controller within the deadline (30 days from the design delivery to the controller).
- *Procedures for documentary checks on projects:* the check takes place mainly on a documental basis, such as the design drawings and the necessary information provided to the controller. The interaction between check activities and the design must take place, through the delivery of intermediate versions of the works, during periodic meetings between the controller, the designer, the OC and the managing body (with the appropriate frequency established by the OC). The documentary check of designs must be supplemented by one or more on-site inspections. The activity of the inspector is shaped as a final inspection report, and possibly also of intermediate reports:

    - name and location of design, design phase, the data of the managing body and the designer, the date of drafting;
    - a summary description of the design and its purpose;
    - a precise description of the documentation contained in the file delivered during the initial meeting with the OC and the designer;
    - specific information regarding the meetings with the OC and the designer (dates of the meetings, reasons for the meetings, participants);
    - information relating to any inspections of the site concerned by the design (dates of inspections, participants);
    - any examination of safety issues that have remained unsolved in previous verification phases;
    - control sheets used;
    - a list of highlighted hazard factors, with analysis of detected safety issues;
    - identification of recommendations with the purpose of eliminating or mitigating potential hazard factors;
    - a summary, also in tabular form, of the problems and their solutions.
- *Modality of the first testing inspection on the design*.
    - infrastructure construction phase: to assess the implementation of the indications contained in the control report of the blueprint design, to propose and to suggest design improvements which were unpredictable or not possible to foresee during the design phase.
    - pre-opening phase to traffic: check the correspondence between the design and the building of the road infrastructure in its physical, geometric and functional standards, before it is travelled by users. It aims to verify:
        - the correct perception and readability of the road infrastructure in all the operating conditions;
        - the intelligibility of intersection;
        - daytime and night-time visibility of horizontal and vertical signs;
        - the efficiency of lighting systems;
        - the adopted traffic regulation, in order to detect any problems in the interference between the different vehicle flows;
        - the presence of unprotected obstacles;
        - the conditions for restraining device installations;
        - the state of the road pavement.
    - *first year of operation*: evaluating the real functioning of the infrastructure from the safety point of view.

Transposition of results from checks

In the event that the design is not adequate according to recommendations contained in the check report, the managing body must justify the reasons for the non-adjustment to the OC. The OC will then have to decide whether to accept the justifications or to order the design adjustment.

- During the *construction and pre-opening to traffic* phases, the results of the relative checks will be contained in the inspection reports, which will be sent to the managing body through the OC, but which must also be promptly transferred to the On-site Works Manager;
- In the *first year of operation*, the purpose of the inspection is to evaluate the real operation of the infrastructure from the safety point of view (crash data) that will allow the OC to evaluate the effectiveness of the carried-out interventions.

5.2.3.4 Check contents

New infrastructures and adjustments of an existing infrastructure

In case of new infrastructure and adjustments of an existing infrastructure:

- the *new infrastructure* must include all necessary safety requirements. The check consists of the analysis aimed at assessing the effectiveness of the designed orographic, environmental, climatic and traffic conditions. Consequently, it enables to highlight those design aspects that should be adequate because they do not comply with the standards, or that could be improved for safety benefits with immediate countermeasures, that the safety expert is able to identify and suggest.
- *Infrastructure adjustment*. The designs are sometimes the result of the application of standard design schemes. These schemes are not specifically adapted to places and conditions, and so they are mainly aimed at solving critical issues (congestion, deviation of the itinerary, variation of local public transport layouts, etc.) different from those related to road safety. Therefore, such problems can even increase road safety problems. The controller must verify that the designer's failure in complying with the rules is justified (insurmountable constraints or costs disproportionate to the intervention). Subsequently, the controller must verify that the design has considered all the safety requirements, through the individuation of the most suitable solutions. This solution must be in line, and in any case compared with those which have given rise to the adjustment intervention.

The suburban and urban environment

In case of suburban and urban environment, the following aspects should be considered.

- *Rural design*. The identification of the aspects to be controlled, is easy to define and consequently also the actual checking activity is more standardizable by type of road and design phase. The aspects to be checked are illustrated in the check sheets.
- *Urban projects*. The variability of situations and the amount of constraints of different nature require a dedicated control to the specific design, which is difficult to be standardized. Therefore, the control of a substantial modification of an urban infrastructure is potentially much more complex and requires particular attention.
- *Segments of rural roads that cross settlements*, even if not strictly related to the definition provided by the D.Lgs. n. 285/1992, and that therefore cross an "urban area", constitute a hybrid and delicate situation from the checking activity point of view. These sections of secondary and local roads should be theoretically adapted to different contexts and modify their technical-functional characteristics accordingly, assuming those of urban roads (route, cross-section, carriageway composition) in order to safely perform a double function.

Dual carriageway and single carriageway roads

The third subdivision of checks on designs refers to the type of road: for these purposes it was considered that a distinction was made only on the basis of the type of carriageway, single and double. In fact, in the rural area, the differences between check contents to be carried out (between type A and type B roads according to the D.M. n. 6792/2001) are minimal.

This minimal difference is justified because technical-functional characteristics provided by design standards and by the D.Lgs. n. 285/1992 between type A (rural) and type B roads are minimal and mainly limited to aspects suchas tolls. Similarly, again in the rural area, the minimum differences between the requirements of type C secondary roads and type F local roads (D.M. n. 6792/2001), both of which have a single carriageway, lead to a substantial equivalence of the respective checks.

The design levels: preliminary, final and blueprint

The main elements to be analysed are different depending on the design stage.

Tab. 5.8: Reference matrix for checks

				New infrastructures				*Adjustment of existing infrastructures*			
				Rural		*Urban*		*Rural*		*Urban*	
				Dual carriage-way	*Single carriage-way*	*Dual carriage-way*	*Single carriage-way*	*Dual carriage-way*	*Single carriage-way*	*Dual carriage-way*	*Single carriage-way*
Design Check Activities	Checks	Preliminary Design	Checklists	x	x	x	x	x	x	x	x
		Final Design		x	x	x	x	x	x	x	x
		Blueprint		x	x	x	x	x	x	x	x

The previous table is the reference matrix for the different possible controls.

- *Preliminary design***.** The main elements to be analysed are:
  - contents of the VISS;
  - geographical framework analysis;
  - relations with the existing road network and with the activities present or planned in the territory in which the infrastructure is inserted;
  - analysis of the horizontal-vertical alignment layout of the new infrastructure (design speed, geometry, number and type of lanes, types of intersections and/or junctions, sight distance checks);
  - traffic type of the new infrastructure;
  - adequacy of the design solution analysed from the point of view of general consistency with the design standard.
- *Final design***.** The safety checks are about the design drawings that illustrate the technical characteristics of the road (report; overall layout; horizontal alignment; elevation profiles; cross sections; standard cross sections of the road body and pavements; main other structures; intersections; sight distance profiles; interferences). The check allows to examine the infrastructure in a stage when it is still possible to change the design choices, considering:
  - introduced changes following the analysis carried out in the preliminary design phase;
  - the vertical-horizontal alignment characteristics of the layout (for an appropriate sizing and coordination of the various elements related to the design speed);
  - analysis of the criteria adopted for the composition of the layout, in order not to undermine the readability of the road environment;
  - the geometry and organization of road spaces, according to the demand and the traffic components;
  - the typological choice and functionality of the intersections and the correct sizing of the individual elements (specialized lanes, accumulation lanes, ramps, etc.);
  - specific analysis of the accesses (minor roads, accesses, etc.);
  - analysis of the service tools (service areas, parking and parking areas);
  - analysis of road restraining systems.
- *Blueprint*. The check allows to examine the dynamic and functional characteristics of the infrastructure:
  - excerpt of the final design;
  - technical report;
  - plan;
  - layout plan;
  - elevation profiles;
  - cross sections;
  - typological road sections of the road body and pavements;
  - main structures;
  - plan views and profiles of interchanges and intersections;
  - plan views and profiles of interferences with underground utilities (public and private);
  - private accesses;

- traffic signs;
- secondary systems (lighting, ventilation, rescue), street furniture;
- maintenance program.

5.2.4 Safety inspection on road infrastructure

5.2.4.1 Purpose of inspections

Safety inspections consist of:

- *diffused_inspections* over the entire homogeneous road segment, including a series of inspections carried out along the road axis and a series of inspections of the singular points of the road axis itself, such as intersections, accesses and segments of particular shape and size, segments next to manufactures, narrowing, barriers, etc.;
- *point or detailed inspections* located on individual critical or potentially critical sites and on singular points.

The purpose of safety inspections is to:

- *identify the criticality of the road infrastructure directly related to crashes;*
- *identify the potential danger factors of the road infrastructure, which could lead to sites with a high number of crashes;*
- *identify the priority of infrastructural corrective interventions to reduce the number and severity of crashes;*
- *identify the priority of infrastructural corrective interventions to prevent further crashes;*
- *keep the safety status of the road network under constant observation.*

Tab. 5.9: Types and characteristics of inspections.

Type Of Inspection	*Where*	*Type Of Crash*	*Purpose*	*When*	*Priority*	*Program*
Diffused	All Network (For Homogeneous Segments)	Distributed (Total Crashes)	Preventive	Periodic	Segments With A Higher Concentration Of Diffuse Crashes	Single Programme Homogeneous Segments + Critical Sites (Network-Wide Safety Classification)
Point	Single Critical Site	Localized (Beforehand Fatal Crashes)	Preventive + Curative	Periodic + Ad Hoc	Sites With More Concentrated Crashes	
	Construction Site		Preventive	Ad Hoc	Higher Traffic Flows	Construction Site Programme

The inspector's analysis is illustrated in a final report in which it is explained whether no critical issues emerge or whether the management or infrastructural measures necessary to improve the safety characteristics of the homogeneous road segment examined must be indicated. The management measures are generally characterized by low construction costs and the possibility of being implemented almost immediately. Infrastructure measures require more in-depth assessment as they have more significant economic commitments and the inspector will also be able to point out various potential corrective measures for them. So, the OC can make an appropriate decision according to the economic assessment of the measures.

5.2.4.2 Stages of the inspection procedure

Implementation of inspection programs

The priority order of inspections is independent from road type because it is linked to the classification of safety, expressed in absolute and non-relative terms by type of road. Consequently, secondary roads could be inspected before roads belonging to the main network, if they had a higher safety potential than the former ones.

The road elements to be inspected are:

- homogeneous road segments, including intersections and all other singular points on the layout;
- the individual critical sites, where there has already been a concentration of crashes, and potentially critical ones, which nevertheless fall within homogeneous segments;
- Construction site.

Moreover, for what concerns:

- *Times*: the OC identifies the inspectors in order to respect the times for carrying out inspections on the entire network. These inspectors will conduct inspections on a first group of more critical homogeneous segments. For the entire program to be completed within two years, the identification of inspectors must be based according to the number of internal inspectors for each OC. Therefore, the inspection program, depending on the internal resources of each OC, must be divided into groups of homogeneous segments by crash metrics values included within certain thresholds set by the OC, so that the latter can assess the need for external inspectors.
- *Mode*: the inspection activity may not be entrusted to a person who has or has had a direct or indirect role in the management and/or design of the infrastructure to be inspected. The OC shall use internal personnel, assigning the inspection activity to personnel not belonging to the OC.

Inspection assignment

The following concerns the inspection assignment:

- *Entrusting object.*
    - *Rural network*: the OC can assign the inspection about one or more homogeneous road segments to the same inspector (or group) or it may decide to split the inspection into homogeneous sub-segments. In order to have an exhaustive assessment, it is appropriate that the OC assigns the single homogeneous segment for both directions of travel for dual carriageways. The diffused inspection also includes the inspection of critical points;
    - *Urban area*: in the case of large urban areas on the primary network (urban freeways and urban roads) and main networks (urban freeways), the inspector's field of activity can still be identified on the basis of the homogeneous road segment. In the case of the secondary and local roads, and of the entire road network of small towns, it seems more useful to constrain the scope of inspections according to further and specific criteria (areas, neighbourhood, etc.).
- *Inspector's responsibility.*

  The OC has accepted the indications from the inspector, after sharing and taking responsibility, and imposes them to the managing body.
    - *Regarding its client*: the OC must respect terms and conditions contained in the engagement letter.
    - *Regarding the manager*: when the inspector believes that the inspection activity could cause negative consequences (slowdowns, traffic closures, dangerous situations due to the concomitance of people and vehicles) he/she will have to properly coordinate his/her tasks with the managing body.
- *Inspection deployment method.*
    - The OC, for each homogeneous segment, identifies a single inspector or multiple inspectors.
    - The inspection on the road segment must be preceded by a meeting between the inspector, the managing body and the OC, which provides information on the conditions of use of the road segment and preliminary data (maps, recently carried out interventions or further implementations, traffic data, crash analysis, crash reports).
    - In the case of the single inspector, the vehicle is generally conducted by a collaborator but in the preliminary inspection, aimed at perceiving globally the road, the vehicle should be driven by the inspector him/herself. In the case of a group of inspectors, their alternation in being drivers and passengers is appropriate, so that all the inspectors can evaluate the road perception.
    - The inspection must be carried out along the road segment *in both directions* of travel in different ways.
    - During inspections**,** *photographic surveys and video recordings* can be carried out.

- The *data georeferencing* must be set, carrying out surveys and measurements with a special vehicle equipped with suitable technological systems (location system with GPS module, digital video cameras, computers capable of recording and processing the images detected).

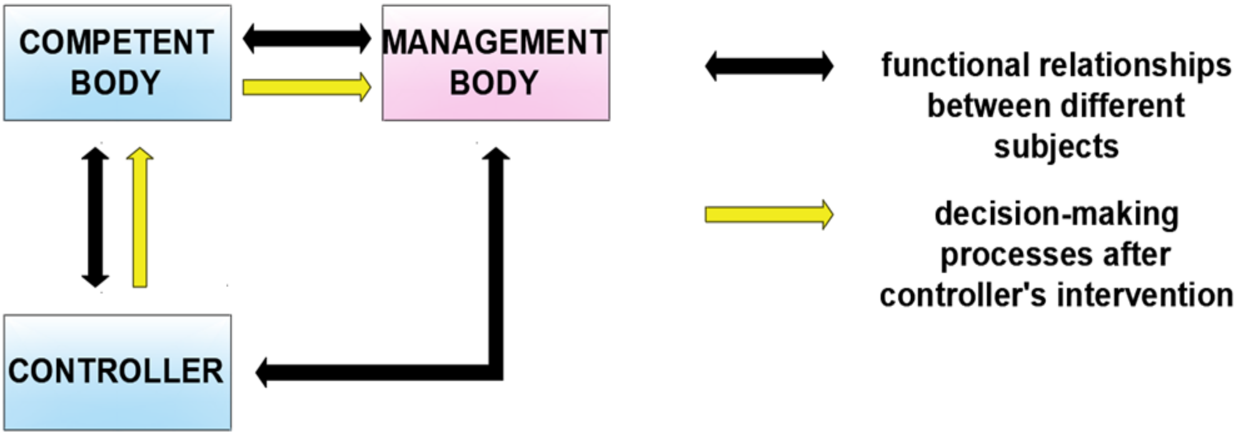

Fig. 5.20: Functional relationships between OC, managing body and inspector.

Inspection of the road segment

Security inspections consist of:

- diffused inspection on the entire homogeneous road segment;
- point inspections located on single critical or potentially critical sites and on singular points.

Diffused inspection

The inspection must be conducted along the road segment (in both travel directions), generally on board of a motor vehicle or, in case of necessity, of other vehicles (motor vehicles, velocipedes, heavy vehicles). The following aspects should be considered.

- *Preliminary inspection*: general problems and characteristics of the road segment according to the entire network, morphology and land use must be detected. The road must be travelled at a speed coherent with the geometric characteristics (*design speed*) in order to assess how the road environment is perceived and interpreted by users. A preliminary night-time inspection must be carried out in which potential further critical issues of the road segment that had not emerged with daylight could be highlighted. The inspector completes the first part of the inspection sheet in which the global notes, for the entire homogeneous road segment, must be reported.
- *General inspection*: safety problems distributed along the entire road segment must be examined in greater depth. The road is travelled at low speed (about 60-80 km/h for "A" and "B" roads and about 30-40 km/h for "C" and "F" roads). For each parameter observed, in the second part of the sheet, the severity of the problem is indicated (M = medium problem; G = serious problem) and it attributes a degree of adequacy/inadequacy to the road segment in a sufficiently objective way. The judgments on the sheets are inserted for segments of variable length. During the journey, a video recording can be taken.
- For neighbourhood and local streets, the inspector will have to identify the most appropriate method for the real characteristics of the road segment, he/she may also include inspections on board of motor vehicles and velocipedes, as well as by foot. In the final part of the inspection sheet, all the references to the point sheets related to each singular point of the layout are reported.
- In conclusion, a general *night-time inspection* must be carried out, using the same daytime inspection form, where any additional critical issues not found during the daytime inspection (signs, street lighting, etc.) must be reported.

Point inspection

It is useful to examine safety problems located in specific sites, where a considerable number of crashes have occurred in proportion to the traffic flow, or which have highlighted critical aspects during the diffused inspection phase.

Furthermore, point inspections are carried out in all singular points of the layout: intersections and other important interferences (tunnels, viaducts, etc.). The point inspections aimed at a detailed analysis of the site, with surveys, measurements and checks, also require an inspection by foot.

The drafting of the final report

In the final report, all the problems, concentrated and widespread, found in the entire road segments are described in a report to which the used inspection sheets are also attached.

The inspector is responsible for identifying one or more possible solutions, without going into the economic aspects.

The inspector must provide not only qualitative but also quantitative indications of the seriousness of the problems, which will consequently correspond to different technical solutions.

The inspection report must be delivered to the OC, dated and signed by the inspector:

- the inspection information (inspection dates) and associated sheets;
- the identification of the prescriptions/recommendations/indications in order to eliminate or mitigate the potential danger factors, with an explanation of the type of benefits that can be achieved through their implementation (e.g. reduction in the number of crashes of a particular type, reduction in the severity of crashes, reduction in the traffic volumes of conflicting flows, etc.);
- possible photographic reports;
- a summary, also in tabular form, of the problems and related solutions, including alternative solutions, separated into prescriptions, recommendations and indications;
- *prescriptions*: solutions to serious lacks related to the failure to comply with primary design and management standards, which must be adopted and implemented by the Managing Body after a specific request by the OC;
- *recommendations*: solutions that contribute effectively to the improvement of safety but require joint planning and programming between the OC and the Managing Body;
- *indications*: suggestions that are easy to be implemented by the managing body.

Tab. 5.10: Essential contents of the final report.

	Problem	*Prescription*	*Advice*	*Indication*
1	Signs not visible at night		remake and adjustment of signs	
2	At-grade intersection after a curve	Speed reduction	adjustment of warning signs	
3	Carriageway reduction due to the road overpass light reduction	Speed reduction	Road overpass reconstruction	
4	Wrong position of a crosswalk in respect of the bus stop		Relocation of crosswalk or bus stop	
5	Wrong design of the access to the intersection			Replacement of parking spots with specialised right-hand lanes

5.2.4.3 Contents of inspections

The reference matrix for the contents of the inspections is reported below.

Tab. 5.11: Inspection reference matrix.

				New infrastructures				*Adjustment of existing infrastructures*			
				Rural		*Urban*		*Rural*		*Urban*	
				Dual carriage-way	*Single carriage-way*	*Dual carriage-way*	*Single carriage-way*	*Dual carriage-way*	*Single carriage-way*	*Dual carriage-way*	*Single carriage-way*
Design Check Activities	First verification: design inspections	Construction	Inspection sheets	x	x	x	x	x	x	x	x
		Pre-opening		x	x	x	x	x	x	x	x
		First year of operation		x	x	x	x	x	x	x	x
Infrastructures Inspective Activities	Inspection at a given speed	Periodic: diffused		x	x	x	x	x	x	x	x
		Periodic: point inspection		x	x	x	x	x	x	x	x
		Extraordinary (construction site): point inspection		x	x	x	x	x	x	x	x

Inspection sheets

The *first part of the sheet* contains data that can be filled in before the inspection (name, number and type of road, length of the road section to be inspected, direction of travel of the section, GPS coordinates at the start and the end of the section, date, time, inspector name, etc.) and other data to be filled in during the preliminary inspection concerning the overall attributes of the homogeneous section.

- Macro-item:
  - "General aspects" (critical environmental conditions, traffic, surrounding landscape, speed, signs).
  - "Geometry" (horizontal alignment, vertical alignment, vertical-horizontal coordination).

The *second part of the sheet* contains a grid that divides the continuous analysis, imposing recurrent checks of the single elements according to the relative indicator, with a variable step or frequency essentially by road type.

- Hyper-headings (macro-items):
  - roadway location;
  - signs;
  - accesses;
  - pavement;
  - lighting;
  - other aspects (variable and specific depending on the scope).

Example of a preliminary diffuse periodic inspection sheet

The following is an excerpt from a preliminary diffuse periodic inspection sheet.

Tab. 5.12: Example of a preliminary diffuse periodic inspection sheet.

Macro Item	*Item*	*Parameter*	*Metric*	*Judgment*
General Aspects	Critical Environmental Conditions	Weather Conditions (Fog, Snow, Wind, Rain)	Lack or Insufficient Warning	
			Inadequacy of Countermeasures	
		Road Surface Conditions (Fog, Snow, Wind, Rain)	Lack or Insufficient Warning	
			Inadequacy of Countermeasures	
	Traffic	Volume	Inadequacy of The Cross-Section	
		Type	Presence of Particular Components	
	Landscape	Zone of Relevance	Presence of Obstacles or Dangers, Presence of Service Roads	
		Buffer Zone	Presence of Buildings, Trees, Etc.	
		Beyond the Buffer Zone	Driving Distraction due to The Context, Adjacent Roads, other Infrastructures, Billboards	
	Speed	Design Speed - Maximum Speed Allowed	Excessive Difference (+ / -)	
		Maximum Speed Allowed - Operating Speed	Excessive Difference (+ / -)	
	Signalling System	Horizontal Signs	Non-Homogeneity	
		Vertical Signs	Non-Homogeneity	
		Variable Message Signs	Ineffectiveness of Information	
Geometry	Horizontal Alignment	Tangent	Excessive Lengths	
		Spiral Transition Curves	Absence or Inadequacy	
		Circular Curves	Inadequate Radii of Curvature	
	Vertical Alignment	Uphill, Downhill Roads and Flat Terrain	Excessive Gradients	
			Excessive Lengths	
		Crest Vertical Curve	Presence of Crests	
		Sag Vertical Curves	Presence of Sags	
	Horizontal-Vertical Coordination	Layout Perception	Visibility Problems	
			Mis-perception of the Layout	

In the following example sheet, each metric is graphically marked with a different colour for medium (“M” - yellow) and severe (“G” – red) judgement.

Tab. 5.13: Example of a filled inspection sheet.

General Inspection												
Macro Item	*Item*	*Parameter*	*Metric*		*Incremental Mileage*							
					0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Road	Platform, Margins and Zone of Relevance	Roadside	Absence or Insufficient Width	M		✓	✓	✓	✓			
				S		✓	✓	✓	✓	✓	✓	
			Restoring in Correspondence of Principal Constructions	M								
				S		✓	✓	✓	✓			
		Emergency Lane	Absence or Insufficient Width	M								
				S								
		Lane or Fast Lane	Insufficient Width	M			✓	✓	✓	✓	✓	
				S			✓	✓			✓	✓
			Excessive Width	M								
				S								
		Internal Shoulder	Absence or Insufficient Width	M		✓	✓	✓	✓			
				S					✓	✓	✓	
		Median	Inadequate Organization of Spaces	M								
				S								
			Negative Effects on Visibility	M				✓	✓	✓	✓	✓
				S	✓	✓						
		Safety Barrier	Absence	M		✓	✓			✓	✓	
				S	✓			✓	✓		✓	✓
			Inadequacy of The Type	M								
				S								
			Inadequacy of Transition and Terminal Zones	M	✓	✓			✓	✓		
				S		✓	✓	✓	✓			✓
			Inadequacy of Traffic Gates	M						✓		
				S								
			Inadequate Installations	M								
				S		✓	✓	✓	✓			
			Presence of Non-Protected Obstacles	M	✓	✓			✓	✓		
				S								
		Scarp	Inefficient Vegetation Maintenance	M								
				S	✓	✓				✓	✓	
			Lack of Safety Barriers	M								
				S				✓	✓			
		Drainage Systems	Inefficient Maintenance	M								
				S							✓	✓
		Fence	Inefficient Maintenance	M				✓	✓			
				S	✓			✓				

The following are the indicators relating to the parameters to be inspected:

- item “*environmental critical conditions*”:
  - parameter: weather conditions;
  - parameter: road pavement conditions;
- item “*traffic*”:
  - parameter: volume;

    - parameter: type;
- item “*landscape*”:
    - parameter: zone of relevance;
    - parameter: buffer zone;
    - parameter: beyond the buffer zone;
- item “*speed*”:
    - parameter: design speed - maximum speed allowed;
    - parameter: maximum speed allowed - operating speed;
- item “*signs*”:
    - parameter: horizontal signs;
    - parameter: vertical signs;
    - parameter: variable message signs;
- item “*horizontal alignment*”:
    - parameter: tangent;
    - parameter: spiral transition curves;
    - parameter: circular curves;
- item “*vertical alignment*”:
    - parameter: downhill, uphill roads and flat terrain;
    - parameter: sag vertical curves;
    - parameter: crest vertical curves;
- item “*horizontal-vertical alignment*”
    - parameter: layout perception.

The last part of the general sheet contains references to the punctual inspections to be carried out in critical points and singular points.

Tab. 5.14: Example of inspection sheet - punctual inspections.

Punctual Inspection Reference									
		Incremental Mileage							
		0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Critical Point	*Critical Point Before Inspection* (Must Be Identified with P1, P2, ... Under the Milepost)		P1						P2
	Critical Point Present During the Inspection (Must Be Identified with Pn+1, Pn+2, ... Continuing the Previous Line, Under the Milepost)			P3	P4		P5		
Singular Points	*Overpasses (Viaducts, Bridges) and Underpasses* (Must Be Identified with V1, V2, ... Under the Milepost)		Ls1			Ls2			Ls3
	Tunnel (Must Be Identified with G1, G2, ... Under the Milepost)								
Construction Site (Must Be Identified with C1, C2, ... Cn ... Under the Milepost)					C1				

5.2.4.4 Inspections on road construction sites

Construction sites are always and in any case a disturbance to flow regularity that lowers safety level and represents a risk factor for crashes. They deserve special attention when they affect roads belonging to the main network, characterized by high operating speeds, even if properly and correctly marked.

The OC, when the managing body notifies the imminent opening of a construction site, may arrange a specific punctual inspection, that is repeated according to the extent of the construction site and its duration, in order to verify that safety regulations are complied.

Given the number of road construction sites, it is impossible for the OC to make inspections on each of them, which is why such inspections are only carried out on significant construction sites (in terms of time duration, importance, size, type and extent of works, degree of dangerousness in relation to flows, crashes that have already occurred).

In particular:

- in *rural areas*, construction sites lasting more than one month must be checked through at least one extraordinary inspection, possibly carried out at the initial stage of preparation or start-up of the construction site itself.
- In *urban areas*, the parameter related to the duration of the construction site must always be set according to the importance of the road, both as a function of the flows and the presence or absence of alternative routes to the road section affected by the works.

The *elements to be assessed* are:

- evaluation of the conformity of the design indications, contained in the specific works, to the real characteristics of the physical sites for "infrastructure projects" which require a specific study of the construction site;
- verification of the implementation of temporary safety measures, according to standard schemes and types which were agreed in advance between the OC and the managing body;
- the adaptability verification of the adopted measures to the specificity of the worksite, the context, and the road type;
- the need to adopt corrective and/or supplementary safety measures;
- the adequacy of signalling systems in accordance to the Ministerial Decree 10/07/2002 (*Technical regulations on signalling schemes, classified by road category, to be adopted for temporary signalling - Disciplinare tecnico relativo agli schemi segnaletici, differenziati per categoria di strada, da adottare per il segnalamento temporaneo*) depending on the road category and the worksite duration.

Construction site in rural areas

The inspections, during both day and night, are carried out by driving a motor vehicle along the road section affected by the construction site. The elements to be assessed are:

- platform;
- road markings;
- vertical signs;
- lighting;
- other elements related to safety and information, such as
  - lighting and visibility at night;
  - *safety measures*.

Construction site in urban areas

Daytime inspections are carried out by driving around the site area on board of a motor vehicle, and possibly by foot, while night inspections are carried out only on board of the motor vehicle. The elements to be assessed are:

- platform;
- road markings;
- vertical signs;
- lighting;
- other elements related to safety and information, such as
  - lighting and visibility at night;
  - *safety measures*.

5.3 Realistic prediction of application times

The extension until 2021 of the application of D.Lgs. n. 35/2011 on roads of national interest stated by the Ministerial Decree n. 586 of 19 December 2019 and the possible difficulties of implementation faced by local authorities suggest that the implementation of the system can be expected in the period 2021 - 2026. Therefore, in recent years there has been no clear procedures about which protocol to follow for existing road safety upgrading projects.

5.4 European standards and strategies for the safety management of the road network

The Directive 2008/96/EC has been transposed into National laws by the various countries of the European Union (in Croatia, after July, 2013). Some countries immediately promoted National Implementing Measures of the Directive, considering also the progressive application of the safety management to the lower road networks, starting from the TEN-T road network. However, some countries already had some road safety management-related procedures in the road safety management system, which were differently adapted after the Directive was issued and locally implemented.

The Road Infrastructure Safety Management system proposed by the Directive 2008/96/EC is based on four main pillars (considering the summary made by Transport&Mobility Leuven in 2014[3], based on Gerlach, 2012[4]), listed below.

- Road Safety Impact Assessments: analyses of the impact of a new road (or a significantly enhanced existing road) on the safety performance of the road network, starting from the initial planning stage until the project approval.
- Road Safety Audits: independent detailed safety checks of the technical aspects of the road infrastructure design with the aim of highlighting safety criticalities, starting from the planning to the early operation stage.
- Road Safety Inspection: routine inspections conducted on existing roads, aimed at highlighting safety criticalities of several road-related aspects, in order to intervene with appropriate maintenance and countermeasures to avoid/minimize traffic crashes.
- Network Safety Management: analysis of the road network with the aim of ranking the various sections based on the large number of crashes occurred compared to the traffic flow and/or based on their safety "potential" related to cost savings, to highlight sites which may mostly benefit from interventions.

Moreover, besides specific guidelines on how to perform assessments, audits, inspections and network management, different European countries adopt high-level safety plans and strategies to reduce traffic crashes. Those strategies may have a specific focus on some aspects in given countries more than other countries.

Based on the publications made by the European Road Safety Observatory (https://ec.europa.eu/transport/road_safety/specialist/erso/country-overviews_en), some essential indications about road safety plans, strategies and required actions are reported below for the countries of the European Union (up to 2019).

After, a specific focus on assessments, audits and inspections in the different EU countries is provided.

5.4.1 European road safety strategies and plans

In several UE countries (up to 2019), a road safety plan was adopted in 2011, covering the period 2011-2020, posing targets mainly related to a decrease in fatalities (i.e., considering the European target of -50% fatalities reduction).

[3] Transport & Mobility Leuven (2014), *Study on the effectiveness and on the improvement of the EU legislative framework on road infrastructure safety management Final Report.* specific contract Move/A3/350-2010 Impact assessments and evaluations (ex-ante, intermediate and ex-post) in the field of transport study on the effectiveness and on the improvement of the EU legislative framework on road infrastructure safety management (Directive 2008/96/EC). Preliminary analysis of some crucial areas for road safety and for safety of road infrastructure. European Commission. Directorate general for Mobility and Transport.

[4] Gerlach J. (2012), *Road infrastructure safety management as part of the Decade of Action for Road Safety. Preconditions, instruments and examples from Europe*. Association for European Transport (AET). European Transport Conference 2012.

In some other countries (i.e., Cyprus, Denmark, Estonia, Finland, Hungary, Ireland, Italy, Latvia, Luxembourg, Malta, Netherlands, Poland, Portugal, Romania, Slovenia), updated plans with different time spans were adopted in years ranging from 2012 to 2016.

In France, the plan is issued and updated annually. In Sweden, there are no traditional high-level road safety plans, while several agencies/stakeholders have set some objectives in line with the "Vision Zero" approach.

Moreover, the following specific road safety country strategies were adopted by some UE countries.

- *Austria*: based on the Safe System Approach, focusing in particular on the reduction of serious injuries from traffic crashes.
- *Bulgaria*: foreseeing participation of public institutions, local authorities, non-governmental organisations, private sector and society, focusing on the relations which negatively affect road safety.
- *Czech Republic*: based on the fact that road safety is both a common right and shared responsibility.
- *Denmark*: based on the "Vision Zero" approach, including the concept of shared responsibility.
- *Finland*: based on the "Vision Zero" approach, including the concept of shared responsibility.
- *Germany*: focused on enabling safe, eco-friendly and sustainable mobility for all road users.
- *Greece*: directed at improving the safety culture of drivers.
- *Italy*: focused on reducing deaths, in particular of vulnerable users.
- *Luxembourg*: based on the "Vision Zero" approach, involving all stakeholders in road safety, with particular reference to the aim of zero deaths and zero serious injuries.
- *Netherlands:* aimed at promoting sustainable road safety, starting from the human being and preferring the preventive approach, based on the following pillars: road functionality, homogeneity of speed and/or direction, physical/social tolerance, road predictability and recognition, behaviour and awareness.
- *Poland:* based on the "Vision Zero" approach.
- *Slovakia:* based on the "Vision Zero" approach.
- *Slovenia:* based on the "Vision Zero" approach, focusing on speeding, drinking and driving, vulnerable road users; working on three levels: political, strategical and professional.
- *Spain:* aimed at fulfilling thirteen quantitative targets and setting eleven different areas of interventions (e.g., setting specific safety performance indicators, besides those related to the whole number of deaths and injuries).
- *Sweden:* based on the safe system "Vision Zero" approach.
- *United Kingdom:* based on setting some forecasted scenarios rather than specific targets.

It is evident how the "Vision Zero" safe system approach, focused on the priority of eliminating/reducing deaths and severe injuries is widespread across Europe. Vulnerable road users are also often a focus of country strategies.

5.4.2 Required infrastructure management procedures in the European Union

As described in this chapter, there are some procedures related to the road safety infrastructure management set by the EU Directive 2008/96/EC. In this section, the state of these actions (mandatory or not) in the legislation of the EU countries is reported, as based on the same reference source cited above (https://ec.europa.eu/transport/road_safety/specialist/erso/country-overviews_en).

The considered actions are: Safety Impact Assessment, Road Safety Audits, Road Safety Inspections, High-risk Site Treatments.

It is evident from the table that most EU Countries perform Road Safety Inspections on a mandatory basis. Safety Audits and High-risk Site Treatments are also very widespread. Mandatory Safety Impact Assessments are instead less frequent.

However, it should be also considered that this situation may have varied over the years in different countries and so this table can be only used as an indication.

Tab. 5.15:Infrastructure management actions in the EU Countries (based on the online publications by the European Road Safety Observatory, https://ec.europa.eu/transport/road_safety/specialist/erso/country-overviews_en).*

EU Member State	*Action*			
	Safety Impact Assessment	*Safety Audit*	*Road Safety Inspection*	*High-risk Site Treatment*
Austria		x	x	x
Belgium	x			
Bulgaria		x	x	x
Croatia				x
Cyprus			x	x
Czech Republic	x	x	x	x
Denmark		x	x	x
Estonia	x**	x**	x**	x
Finland		x	x	
France		x	x	x
Germany		x***	x	x
Greece		x	x	x
Hungary	x	x	x	x
Ireland	x	x	x	x
Italy		x	x	x
Latvia	x	x	x	x
Lithuania				x
Luxembourg	x		x	
Malta		x	x	
Netherlands	x	x	x	x
Poland	x	x	x	x
Portugal				x
Romania		x	x	
Slovakia		x	x	x
Slovenia		x	x	x
Spain		x***	x	x
Sweden	x	x	x	
United Kingdom		x	x	x

*The most recent data referred in these sources are from 2016 (for several countries data are older, up to 2010).

**Only for TEN-T road network.

***Only for federal state level projects.

5.5 References

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Transport & Mobility Leuven. (2014), *Study on the effectiveness and on the improvement of the EU legislative framework on road infrastructure safety management*, *Final Report*. specific contract Move/A3/350-2010 Impact assessments and evaluations (ex-ante, intermediate and ex-post) in the field of transport study on the effectiveness and on the improvement of the EU legislative framework on road infrastructure safety management (Directive 2008/96/EC). Preliminary analysis of some crucial areas for road safety and for safety of road infrastructure. European Commission. Directorate general for Mobility and Transport.

6. Local project protocol and related issues

In the meanwhile, in countries which will stay for a long-time interval without clear indications on how to perform safety interventions on existing roads, what could be done to design road safety adjustments?

While this problem could potentially be applicable in different local contexts, the Italian case is considered here for the sake of coherence with the design applications further presented.

In Italy, the *Guidelines for road safety analysis – Linee guida per le analisi di sicurezza delle strade* have been used as the target standard in road safety analysis during the transitory phase between the European rules (2008/96/CE) and the adoption of national guidelines about road safety analysis. This procedural step is stated indeed by the art. 12, c.5 from the D.Lgs. n. 35/2011, i.e., the regulatory document which converted the European standard guidelines 2008/96/CE about the road safety into Italian law. The new national guidelines, in terms of road safety analysis, have been introduced in the national framework about the whole design procedures of road infrastructures by the D.M. 2 May 2012 n. 137, *Guidelines for the management of road infrastructure safety – Linee Guida per la gestione della sicurezza delle infrastrutture stradali.*

Hence, according to the new national guidelines aimed at road safety, D.Lgs. n. 35/2011, the road safety adjustments on existing roads should be made up to 2020 and later, following the dispositions published in 2001 and thought and written earlier than 2000. But how?

The Italian standards do not provide the “black spot” definitions, neither assess standard thresholds to the crash frequency on roads.

Which metrics should be used? Density? Crash rate? Other?

Which dimension should be chosen for the road segment length? 100 m? 400 m? 1000 m?

Which is he warning threshold for the crash occurrence?

Which is the statistical framework?

Mainly, after having identified the main points, how to choose the intervention according to the crashes occurred?

Unfortunately, nowadays there are no reliable and definite intervention protocols. On the contrary, in the last recent years methods and procedures aimed at solving this problem have been implemented abroad.

6.1 References

Circular of the Ministry of Public Works n. 3699, 8th of June 2001: *Guidelines for road safety analysis –Linee guida per le analisi di sicurezza delle strade.*

Directive 2008/96/EC on road infrastructure safety management, European Parliament and Council;

Legislative Decree 15 March 2011 n. 35: *Actuation of the European directive 2008/96/CE about the road infrastructure management - Attuazione della direttiva 2008/96/CE sulla gestione della sicurezza delle infrastrutture stradali.*

7. The HSM method

In the Italian version of the book, in the Chapter 7, part of the Highway Safety Manual (2010)[1] was translated, describing the method contained therein for road safety assessments. In order to avoid the redundancy of a section entirely devoted to the description of the HSM manual (available in the original language), a summary of the method and concepts proposed in the HSM is provided in the following chapter. For more detailed information, please consult the HSM (2010)[1].

It is important to underline from the beginning that the main innovation proposed by the HSM method consists of a quantitative assessment of both the existing safety conditions on site, and the different possible scenarios of intervention on existing road sites, in the different steps of the road infrastructure safety management process. The quantitative assessment is based on the predictive method, the main features of this method are highlighted in this chapter. The HSM method is applied and integrated with other sources in the case studies presented in the Chapters 13 and 14.

7.1 Road infrastructure safety management and the crash phenomenon

The HSM provides an operational framework for each step of the road safety management process. This process requires continuous monitoring of the reference road network, selection of sites to be improved, identification and assessment of the possible interventions[1]. These operations are closely linked to the quantification of the number of crashes and the people involved in fatal and injury accidents, occurring during a given period, both considering past (observed data) and future (predictions for different project scenarios).

All interventions aimed at reducing crashes should be designed after having carefully considered the causes leading to the observed crashes. However, this process is complex since, as reported in the HSM, in the vast majority of cases there is not a single “cause”, but a wide list of different possible crash contributing factors.

Depending on the objectives set for the road safety study, the analysis of crashes can vary from a purely macroscopic scenario (i.e., an area-wide study) to more microscopic scenarios (a single segment or intersection). However, a crash analysis should always take into account the following factors:

- road infrastructure (geometric, functional and structural characteristics);
- surrounding environment (weather, traffic conditions, reference context, etc.);
- vehicle (type, maintenance status, vehicle-pavement interaction, etc.);
- human factors (user driving behaviour, psycho-physical conditions, age, etc.).

Each of these factors can play a more or less important role in the occurrence of crashes. A useful tool to relate the series of crash-leading events to the categories of factors that contribute to the crash is the Haddon Matrix (1972)[2], which is recalled in the HSM. In this perspective, the "crash" event clearly depends on several variables that are often concomitant and difficult to detect, unless extremely accurate databases and accurate reconstructions of the crash are available. Moreover, theoretically, all possible variables belonging to the different components of the system (or a representative number of them) should be considered when developing and applying crash prediction techniques.

[1] AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

[2] Haddon W. (1972), “A logical framework for categorizing highway safety phenomena and activity”, *Journa-l of Trauma and Acute Care Surgery,* 12(3), 193-207.

7.2 The predictive method

The predictive method is a crucial element in the application of the steps described in the HSM manual (2010)[1] (detailed in the next section). The predictive method estimates the mean crash frequency (for total crashes, or for a given severity level or type of collision) of a site, infrastructure or road network, for a given period, for a given geometric design, traffic volume (AADT) and traffic control characteristics[1].

In particular, such method assesses the mean "expected" crash frequency, which considers both observed crashes (crash data, if available) and predicted crashes thanks to a crash prediction model (Safety Performance Function -SPF-). These predicted models link the crash frequency to geometrical (horizontal/vertical alignment, cross-section, etc.) and functional (average daily traffic, traffic control, etc.) characteristics of the road segment or intersection. The above-mentioned SPF functions should be determined for each type of road element (road with two or more lanes, in urban or rural areas, for segments or intersections, etc.). Actually, there are databases collecting SPF functions already determined for different geographical contexts and for different types of road elements (e.g. the database created by the European project PRACT[3]). An example of SPF is given below:

$$N_{spf\ x} = AADT \times L \times 365 \times 10^{-6} \times e^{(k)} \qquad \text{(Eq. 7-1)}$$

Where,

- $N_{spf\,x}$ = estimation of the predicted mean crash frequency for the SPF related to basic conditions, for a generic road element: "x" segment or intersection (crashes/year);
- AADT = annual average daily traffic (vehicles/day), referring to the road element (in the case of intersections, traffic volumes on the main and secondary roads may be considered separately);
- L = length of the road segment (miles, in the case of HSM or km), variable which is absent in the case of intersections;
- k = coefficient to be estimated from the model regression.

The SPF functions proposed by the HSM manual are structured to obtain a basic crash estimate ($N_{Predicted\_SPF}$) depending only on the length of segments (for road segments) and traffic volumes. This estimate is then multiplied by the Crash Modification Factors (CMFs). These factors take into account differences between the geometric and functional conditions of the site under analysis (generic conditions "b") compared to the conditions considered while developing the basic SPF functions (baseline conditions "a", specifically defined in the HSM). Such differences are then defined as the variation of the mean expected crash frequency of a site from condition "a" to condition "b"[4].

$$CMF = \frac{Mean\ expected\ crash\ frequency\ in\ condition\ b}{Mean\ expected\ crash\ frequency\ in\ condition\ a} \qquad \text{(Eq. 7-2)}$$

The concept of CMFs can also be applied to compare different intervention alternatives (in this case, the conditions "b") compared to a specific base condition (of no intervention, conditions "a"). CMFs lower than 1 indicate that the intervention/condition reduces the estimated value of the mean crash frequency compared to the baseline condition. CMF values greater than 1.00 indicate the opposite, that is an intervention/condition which increases the mean expected crash frequency.

The HSM predictive method assumes that CMFs can be multiplied to estimate the total effect produced by the combination of different interventions/conditions different than the baseline. This approach implies that the effects of each change/adjustment should be independent on the others. Otherwise, similar effects may be computed multiple times, thus affecting estimates.

The predictive method also considers the application of a local calibration coefficient (*Cc*), which takes into account the different context in which the SPF is applied (which is referred to both different geographical contexts and different reference periods, for more detailed explanations see chapter 12). The predicted mean crash frequency ($N_{Predicted\_CMF}$) according to the HSM manual is therefore calculated as follows.

[3] www.practproject.eu

[4] Hauer E. (2000), "Accident modification functions in road safety", *Proceedings of the 28th Annual Conference of the Canadian Society for Civil Engineering*, London, Ontario, Canada.

$$N_{Predicted\_CMF} = N_{Predicted\_SPF} \cdot (CMF_1 \cdot CMF_2 \cdot \ldots \cdot CMF_n) \cdot C_c \quad \text{(Eq. 7-3)}$$

The weighting approach for the predicted mean crash frequency ($N_{Predicted}$), calculated as indicated by equation 7-3, with the observed crashes ($N_{Observed}$) is performed by means of the Empirical Bayesian method (EB). The EB method allows to improve the statistical reliability of the estimate. This method, which is based on the following equations, allows to obtain a value of expected mean crash frequency ($N_{Expected}$), by assigning a weight (w) to the predicted mean crash frequency, which depends on the overdispersion of the model used, the years and the number of crashes observed.

$$N_{Expected} = w\, N_{predicted} + (1 - w)\, N_{observed} \quad \text{(Eq. 7-4)}$$

Where:

$$w = \frac{1}{1 + k \times (\sum_{All\ years\ of\ observations} N_{predicted})} \quad \text{(Eq. 7-5)}$$

k = SPF overdispersion parameter.

The EB method therefore succeeds in estimating the mean crash frequency by giving more weight to data that show greater statistical reliability or to relevant values of observed crashes, in order to obtain a reliable estimate. Therefore, the application of the methodologies described in the HSM manual (2010)[1] requires the availability of local SPF functions or at least of suitable calibration coefficients for the types of roads under examination (to be applied to the functions provided in the HSM manual, 2010[1]).

It should be noted that the general application of the HSM predictive method can in any case be conducted regardless of the origin of the SPF (native HSM or locally developed). Indeed, the manual itself encourages the development of local SPFs to be introduced into the predictive method for applications in other contexts (see Chapter 12).

7.3 Steps of the HSM method

The HSM relies on a continuous and iterative safety management process consisting of several steps.

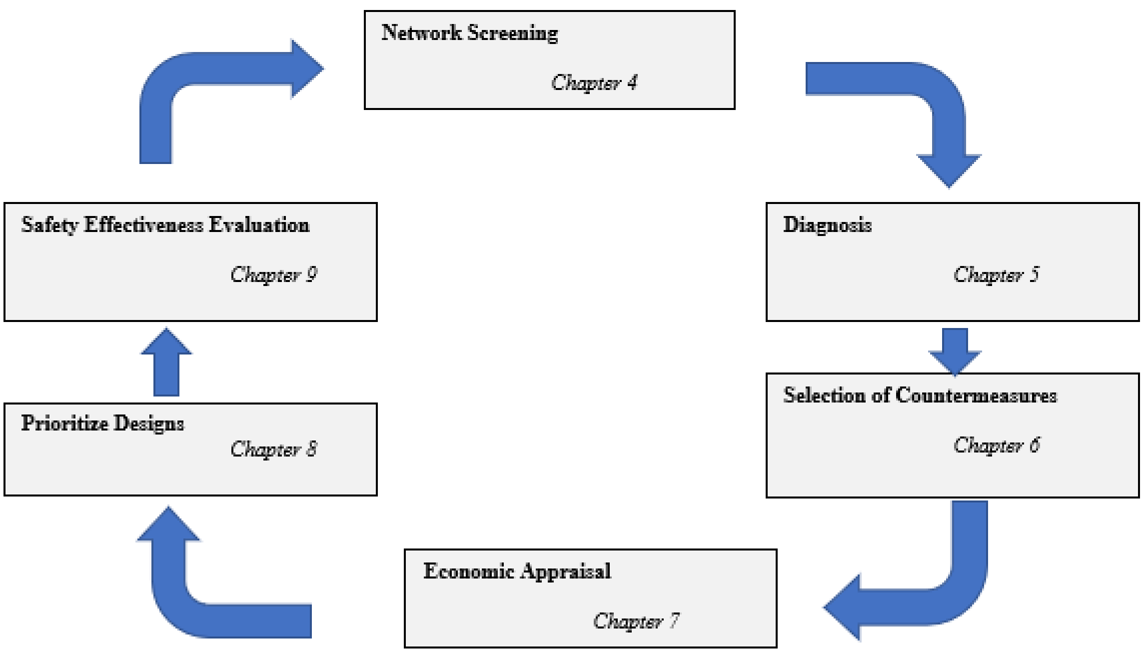

Fig. 7.1: Macro-activities of the road infrastructure safety management cycle (based on HSM, 2010[1]).

The different phases indicated in the figure are briefly described below.

7.3.1 Network screening

The road safety management according to the HSM (2010)[1] includes a preliminary Network screening phase. The HSM network screening is defined as a process able to identify and classify, within a road network, the sites that are characterized by the highest crash reduction potential, by means of the adoption of countermeasures. The screening methods described in the HSM (2010) are based on simple classifications, analyses based on a sliding window or based on the search for peak values. Each of these methods can be useful depending on the objectives to be pursued.

However, it is important, at this stage, to define an indicator for measuring crashes at a road site or the potential decrease/increase in the number of crashes. Therefore, the HSM (2010) proposes a set of crash metrics, whose choice depends on the following factors:

- data availability;
- regression to the mean (RTM) bias;
- how reference thresholds are set.

The thresholds allow to determine the deviation of the calculated indicator from that threshold. The proposed metrics are summarized below:

- mean crash frequency;
- crash rate;
- equivalent mean crash frequency (i.e., referred to PDO crashes, also with EB adjustments);
- relative severity Index (RSI);
- critical rate;
- excess of mean crash frequency (both predicted, or expected, using SPFs or EB adjustments);
- excess proportions of specific crash types (or probability of exceeding threshold proportions);
- Level of Service of Safety (see Chapter 10).

There are several metrics with as many advantages and disadvantages. If it is not possible to obtain detailed data on the network, a preliminary classification can be made using the frequency and the crash rate. However, due to possible errors arising from their use, it is always preferable to use more refined indicators. For a detailed discussion about the indicators, please refer to Chapter 8, where they are compared with the indications provided by the Italian Guidelines.

7.3.2 Diagnosis

This phase deals with all the activities required to understand the causes of crashes and the analysis of boundary conditions. It is crucial as a preliminary stage for the subsequent selection of countermeasures. According to the HSM, the diagnosis process includes the following phases:

1. analysis of safety data: type of crash, severity, identification of the boundary conditions that have contributed to cause the crash;
2. evaluation of any supporting documentation;
3. assessment of on-site conditions: surveys, on-site inspections.

The final output of this phase includes condition diagrams, collision diagrams, inspection reports and the analysis of boundary conditions.

7.3.3 Selection of countermeasures

A countermeasure is the implementation of a physical change in the infrastructure, the control system or of a change in high-level policies aimed at decreasing the crash frequency and/or crash severity at a given site. The HSM (2010) does not explicitly consider vehicle or driver-related countermeasures, but suggests infrastructure-related changes which however could have positive effects on the drivers' perception of the surrounding driving environment.

The identification of crash contributing factors is a complex phase for the choice of countermeasures to be implemented. The Haddon matrix[2], as already mentioned in 7.1, can facilitate and outline this process.

The countermeasures could be proposed according to their role of counteracting or eliminating the problems encountered during the diagnosis phase. The hypotheses that several solutions for the same problem are possible or that a single countermeasure can solve several causes of the crash phenomenon should be anyway considered.

7.3.4 Economical appraisal

The evaluation of the countermeasures adopted in economic terms can actually be an element of selection (and classification) of the possible countermeasures to be implemented.

The economic assessment can be made either by a cost-benefit analysis or by a cost-effectiveness analysis. In both cases, the benefits provided by a countermeasure (or set of countermeasures) can be computed.

In the cost-benefit analysis, the expected variation of mean crash frequency (eventually considering severity) is converted into a monetary value and compared with the cost due to the implementation of the countermeasure (or set of countermeasures).

In the cost-effectiveness analysis, the variation in the crash frequency is instead directly compared with the cost due to the implementation of the countermeasure.

The summary of possible indicators listed in the HSM to economically assess possible alternative countermeasures is provided as follows:

- design alternative costs;
- monetary value of the benefits related to the design alternative;
- reduction in the total number of crashes;
- reduction in the number of fatal and injury crashes;
- Net Present Value (NPV);
- Benefit-Cost Ratio (BCR);
- Cost-Effectiveness index.

A reliable classification that takes into account one or more of the above variables can help decision makers to choose the most appropriate countermeasure or set of countermeasures for the site of interest and possibly considering the economic constraints imposed.

7.3.5 Prioritise designs

According to the HSM, hierarchical methods for prioritizing designs are mainly applicable for large scale programs on multiple sites or for an entire road network, but they can also be useful for comparing design alternatives for a given site.

These methods, listed in the HSM, are summarized below:

- ranking by economic effectiveness measures, this is the simplest method, effective only when the comparison is made for a limited number of sites or countermeasures; it does not take into account budgets or other elements relevant for the project.
- Ranking by incremental cost-benefit analysis, a ranking is carried out identifying the best economic investment by subsequently comparing and ranking the different alternative countermeasures; even if budget constraints are not explicitly considered.
- Optimization methods, the method to identify the countermeasure that leads to the greatest benefits once a given budget is set, eventually used at a macroscopic level to analyse the entire road network; it is however assumed that all the considered projects have been considered as economically viable.

The results from these hierarchical methods can be a key (but not exhaustive) element in the decision-making phase. In fact, other factors besides economic ones can influence the choice of one or more countermeasures.

7.3.6 Safety effectiveness evaluation

The evaluation of the effectiveness of a countermeasure is a process that quantitatively estimates how a countermeasure/set of countermeasures actually modify the crash frequency or crash severity. It is therefore a process that is conducted following the implementation of one or more countermeasures at given sites.

The evaluation is more complex than a simple comparison between the data available before and after the countermeasure, since many crash contributing factors can vary over time, regardless of the infrastructure-related changes. The analyses that can be conducted for evaluation purposes are:

- observational before/after analysis;
- experimental before/after analysis;
- cross-sectional observational analysis.

The observational studies base their estimates on the effects that some countermeasures have had at similar sites. On the other hand, the experimental studies, evaluate the effect of the selected countermeasures on the specific sites under investigation.

7.4 Advantages of the HSM method

The predictive method proposed by the HSM (described in Chapter 7.2) can be used for the purposes of the activities included in the road safety management process listed above to obtain quantitative estimates of crashes at the sites under consideration.

The advantages of the HSM predictive method, in any of its employments in the various steps of the road safety management, are as follows[1]:

- it focuses on the long-term assessment of the expected mean crash frequency rather than observed crash frequency (typically related to short-term periods), so overcoming the RTM bias;
- the uncertainty due to limited availability of crash data for the generic site is reduced by applying predictive models (also non-linear relationships) based on data from similar sites;
- SPF models in the HSM are based on the negative binomial distribution, which is more suitable for modelling the variability of rare crash events than traditional models;

Moreover, the use of SPF models can provide crash estimations also for sites that have not been built (or very recently built) and for which observed data are unavailable.

7.5 References

AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

Haddon W. (1972), “A logical framework for categorizing highway safety phenomena and activity”, *Journal of Trauma and Acute Care Surgery*, 12(3), 193-207.

Hauer E. (2000), “Accident modification functions in road safety”, *Proceedings of the 28th Annual Conference of the Canadian Society for Civil Engineering*, London, Ontario, Canada.

PRACT (Predicting Road ACidents - a Transferable methodology across Europe) Project. www.practproject.eu

8. Comparison between Guidelines and HSM

In the previous chapters, an examination of the Italian Guidelines[1] (based on the European legislation, henceforth referred to as "Guidelines") and the HSM manual (2010)[2] has been presented. Therefore, a critical comparison of the analysed methods/procedures could be attempted in order to develop an operational protocol for interventions. The two methods show similarities but also significant differences, as illustrated below.

8.1 The crash prediction problem

8.1.1 Main definitions

8.1.1.1 Definition of the "crash" event

In the HSM, a crash is defined as a set of events that causes injury or damage because at least one motor vehicle was involved in a collision. It may involve either another motor vehicle, a cyclist, a pedestrian, an animal or an object. Therefore, accidents between cyclists and pedestrians, or rail vehicles, are excluded.

It is important to note that, while the definition itself of crash can be largely similar worldwide, in Italy (and generally in Europe), only fatal and injury crashes are collected and stored in National database. This means that the comparability between datasets and related applications can be troubling.

8.1.1.2 Definition of crash frequency

In the HSM as well as in several other sources, the crash frequency is defined as the number of crashes occurred at a particular road site/section (or even entire facility/network) during a year. It is calculated according to the following equation and it is measured in crashes per year.

Crash frequency [crashes/year] = number of crashes/period [years] (Eq. 8-1)

It is worth to note that in the Guidelines, the frequency is defined with reference to a kilometre of the road, rather than a unit of time, for classification purposes of different road sections.

8.1.1.3 Definition of crash severity

Crashes may have different severities, depending on the level of injury or property damage caused.

[1] Ministerial Decree n. 137 of 2 May 2012: *Guidelines for the management of road infrastructure safety pursuant to art. 8 of Legislative Decree no. 35 of 15 March 2011 - Linee guida per la gestione della sicurezza delle infrastrutture stradali ai sensi dell'art. 8 del decreto legislativo 15 marzo 2011, n. 35.*

[2] AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

In the HSM, the scale used is divided into five severity levels, the so called KABCO scale:

- K - Fatal injury: resulting in death;
- A - Incapacitating injury: injury that prevents the subject from walking, driving, or continuing to perform his/her normal activities as before the crash;
- B - Clearly non-incapacitating injury: any evident injury (different than incapacitating or fatal);
- C - Possible injury: any claimed injury which is not evident;
- O - property damage only (PDO).

Note that the KABCO scale is a synthetic severity scale, while several other possible classifications are present, in particular for the different injury levels. For instance[3], the AIS (Abbreviated Injury Scale) scale is often used in Europe for classifying injuries, which measures the severity from 1 (*slight injury*) to 6 (*non-treatable usually fatal injury*) for each 9 regions of the human body where the injury occurred. The related MAIS (Maximum Abbreviated Injury Scale), which is simply the highest AIS-score of all the injuries on the individual is also used, due to its simplicity. However, the different European countries have several different ways of defining a "serious" injury, even considering a scale.

In some countries, such as Italy, the injury scale is not even defined in official crash statistics so that a simplified scale is produced: fatal, injury or PDO crashes (the latter not recorded in the official National database). This is reflected in the Guidelines, where there is no reference to serious or slight injuries. In other EU countries, at least a distinction between severe and slight injuries can be found in some instances.

8.1.2 The Guidelines are based only on observed crashes

In the Guidelines, the crash data are represented in the following table, with the same notation above.

Tab. 8.1: Representation of the crash data in the Guidelines[1].

Homogenous segment	*Length*	*Deaths*	*Injured*	*Crashes*	*Annual average flow*	*Total travelled km*
	Km	*n*	*n*	*n*	*n (million vehicles)*	*n (million vehicles km)*
A	5	1	3	5	3	15
B	3	1	3	5	6	18
C	2	1	3	5	4	8

For the classification of segments with a high concentration of crashes, crash metrics are considered. Note that the crash frequency is referred to a kilometre of road.

Tab. 8.2: Representation of crash metrics in the Guidelines[1].

Metrics		*(a)*	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*	
Homogeneous segment	*Length*	*Death rate*	*Deaths frequency*	*Death*	*Injury rate*	*Injury frequency*	*Injuries*	*Classification based on Metric (a)*
	km	*N. deaths/ 10^6 veic.*km*	*N. deaths/ km*	*N.*	*N. deaths/ 10^6 veic.*km*	*N. injuries/ km*	*N.*	
A	5	1/15	1/5	1	3/15	3/5	3	2
B	3	1/18	1/3	1	3/18	3/3	3	3
C	2	1/8	1/2	1	3/8	3/2	3	1

[3] European Commission. European Road Safety Observatory. (2015), *Serious Injuries*. https://ec.europa.eu/transport/road_safety/sites/roadsafety/files/ersosynthesis2015-seriousinjuries25_en.pdf

The Guidelines are evidently based only on Observed crashes. This choice appears questionable because, as noted in the HSM:

- crashes are random events and then the observed frequency varies over time. For this reason, the short-term observed crash frequency cannot be used for long-term predictions, given the same boundary conditions.
- The mean long-term crash frequency could only be calculated if all geometric and traffic conditions on a road stay constant, which is a very rare situation, thus requiring an estimate of long-term frequencies.

The unreliability of the estimates of short-term mean observed crash frequencies, due to their natural random variation, is shown in next figure.

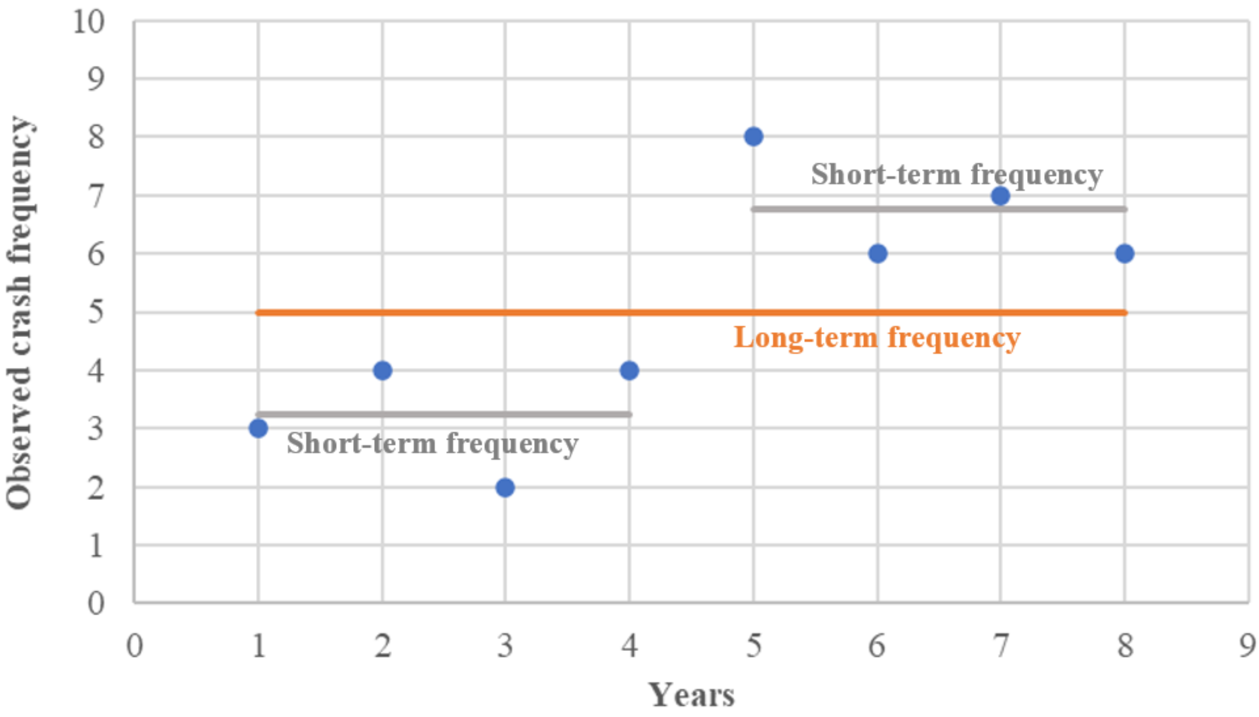

Fig. 8.1: Difference between long and short term mean frequency (based on the HSM manual, 2010[2]).

The effect shown in the previous figure is known as the Regression-To-the-Mean (RTM)[2]: the crash frequency can fluctuate over time between high and low values (generating different short-term average frequencies), but it converges on a long-term average value. Hence, based on this effect, long-term period mean crash frequencies should be estimated (to compare sites or also to assess the effect of given countermeasures). Otherwise, a RTM "*bias*" can affect estimates.

8.1.2.1 EB method (HSM) to overcome short-term variability and site changes

Two main issues were highlighted in the previous sub-section: the short-term frequency variability and the changes in road site conditions; which lead to consider the mean observed frequency as a potentially biased estimator (due to the RTM effect).

These issues can be overcome by using the predictive method introduced by the HSM to estimate the mean expected frequency. This prediction can be referred to different years, so considering changes in road sites over years through site-specific corrections and it can be used to correct likely biased short-term crash frequency observations. In fact, the HSM focuses on estimating the "*mean expected crash frequency*":

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \quad \text{(Eq. 8-2)}$$

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Where:

- $N_{expected}$ = mean expected crash frequency in the study period;
- $N_{predicted}$ = mean predicted crash frequency from a model in the study period and for given conditions;
- w = weighting factor depending on the model reliability and the crash data;
- $N_{observed}$ = mean observed crash frequency of the analysed site.

The Empirical Bayesian EB method is used to weight the mean predicted crash frequency with the mean observed crash frequency, to obtain the mean expected crash frequency.

8.1.2.2 HSM method fundamentals: Safety Performance Functions and Crash Modification Factors

As shown in the previous chapter, the HSM relies on regression models developed as based on data from several sites of a given road type, essential for the application of the EB method.

These models (Safety Performance Functions, SPFs), which are obtained in some specific basic geometric and traffic control conditions, mainly relate the traffic volume AADT to the crash frequency. The output of an SPF is the mean predicted crash frequency ($N_{predicted}$) for a site. A correction of the SPF forecast, in order to consider the difference between the baseline conditions of the SPF and the site-specific conditions is made through the application of the Crash Modification Factors (CMFs). If site conditions are equal to baseline SPF conditions, CMFs are equal to 1.

With reference to the SPF, the crash rate (crash frequency per number of vehicles crossing the road section in the same period) is graphically represented by the slope of the line joining the origin with the points of the SPF (see next figure).

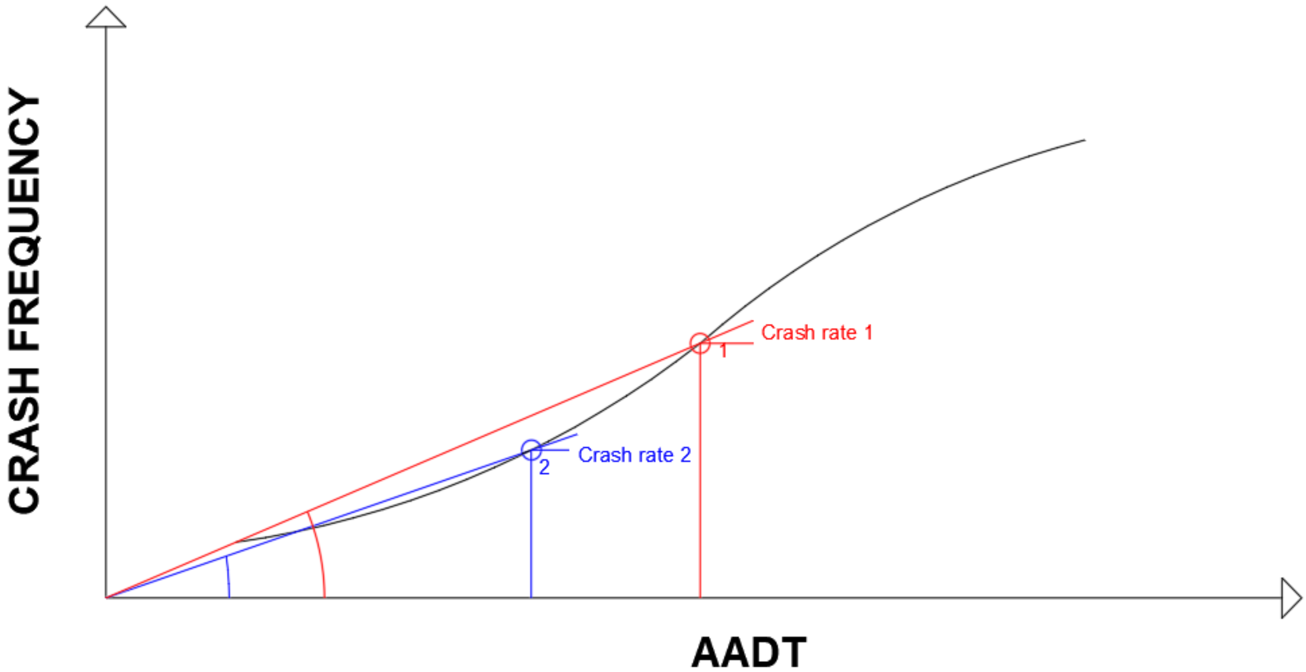

Fig. 8.2: Safety Performance Function and crash rates (based on Kononov and Allery, 2003[4]).

The definition of the predicted crash frequency, including the fundamental elements introduced in the HSM, is then obtained as follows:

[4] Kononov J., Allery B. (2003), "Level of service of safety: conceptual blueprint and analytical framework", *Transportation research record*, 1840(1), 57-66.

$$N_{predicted} = N_{spf\ x} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) \times C_c \quad \text{(Eq. 8-3)}$$

In which:

- $N_{predicted}$ = Mean expected crash frequency for a specific year at a given site (crashes/year);
- $N_{spf\ x}$ = Mean expected crash frequency determined for the basic conditions through the Safety Performance Function representing the road type of the site "x" (crashes/year);
- CMF_{1x} ... CMF_{xy} = Crash modification factors, which are site-specific (referred to the site "x");
- C_c = Calibration coefficient, to consider local conditions.

In detail, the calibration coefficient (Cc) is applied to take into account differences between jurisdictions and periods in which SPFs are developed and applied. In this way, the HSM predictive model can be transferred to other countries (see Chapter 12) and to different time periods.

8.2 Crash metrics problems

In order to determine a hierarchical ranking of road segments and intersections for safety management purposes (e.g., to carry out inspections):

- the Guidelines use the Crash Rate and the ranking is done with the Safety Potential (SAPO).
- The HSM provides many metrics. Each of them is used according to specific objectives.

8.2.1 The Guidelines use the crash rate and ranking is done with the Safety Potential (SAPO)

The Guidelines suggest to use two simple metrics such as the crash frequency and the crash rate to measure safety performances.

The crash frequency was defined at the beginning of this section, while the crash rate is simply the crash frequency related to the exposure in the same reference period. Generally, the number of million vehicles which crossed a given road section or entered into an intersection are used as a measure of exposure to crashes at, namely, segments or intersections, as indicated in the following equation:

$$Crash\ Rate = \frac{Mean\ Crash\ Frequency\ in\ a\ given\ time\ period\ (related\ to\ a\ km\ of\ road\ or\ to\ an\ intersection)}{Million\ vehicles\ in\ the\ same\ time\ period} \quad \text{(Eq. 8-4)}$$

However, as highlighted in the HSM, the use of crash rate as a metric assumes a linear relationship between crash frequency and exposure. However, while crash frequency and traffic volumes are clearly related, in several cases, the relationship is not linear (see e.g. Figure 8.2).

In the Guidelines, the classification of the road network safety is conducted by ranking the segments of the existing road network, according to the effectiveness of the possible intervention, that is according to their potential for improving safety and for saving crash costs.

The Safety Potential (SAPO) metric can be used to prioritize inspections on the different homogeneous network segments.

*SAPO = DCI - BDCI (k€/km*year)* (Eq. 8-5)

Where:

- DCI = crash cost mean density = CAI / L
    - CAI (k€/year) = annual mean crash cost = (Nm*Cm + NfG*CfG + NfL*CfL)
        - Nm, NfG e NfL are the number of dead, seriously and slightly injured
        - Cm, CfG e CfL (k€) are mean death, serious and minor injuries costs
        - L(km) = road segment length
- BDCI = base value of the mean crash density cost = (BTCI*365*TGM)/10^6
    - BTCI (€/1000*vehicles*km) = base crash cost rate
    - TGM (vehicles/day) =annual average daily traffic.

8.2.2 The HSM provides many metrics. Each of them is used according to specific objectives

Crash metrics are classified by the HSM Manual according to available data and their advantages and their disadvantages. According to the HSM, sites can be ranked based on the following metrics (performance indicators):

- mean crash frequency (eventually classified by type or severity);
- crash rate (eventually by type or severity);
- equivalent property damage only (EPDO) mean crash frequency, that is assigning different weight to crashes according to their severity i.e., with reference to property damage only crash costs;
- relative severity index, calculated by referring the average costs of the crashes occurred at a given site to the average crash costs of the reference population (the most dangerous sites show crash costs much higher than the average);
- critical rate, a specific rate different for each site and dependent on statistical considerations (the most dangerous sites show rates largely exceeding the computed reference critical rates);
- excess predicted mean crash frequency using method of moment, the excess is the difference between the mean crash frequency of the reference population and the mean frequency of the given site, corrected by variance and global mean (the most dangerous sites show the highest excesses);
- Level of Service of Safety (LOSS), four LOSS classes are identified depending on the difference between the observed crash frequency at the site and the predicted mean crash frequency in the population sample[4,5] (the most dangerous sites are in the fourth LOSS class);
- excess predicted mean crash frequency using SPFs, the excess is the difference between the observed mean crash frequency at the site and the mean crash frequency by a local SPF (the most dangerous sites show the highest excesses);
- probability of specific crash type exceeding threshold proportion, the excess depends on the probability that the crash type proportion at a given site exceeds the threshold proportion valid for the reference population[6] (the most dangerous sites with respect to the given crash type show the highest excesses);
- excess proportions of specific crash types: similar to the previous indicator, except than the excess is a proportion (not considering probabilities);
- expected mean crash frequency with EB adjustments, the most dangerous sites show the highest expected mean crash frequencies (obtained through the EB method);
- equivalent property damage only (EPDO) mean crash frequency with EB adjustments, the most dangerous sites show the highest expected mean crash frequencies (crash prediction is carried out using the EB method for different levels of severity and converted to PDO crashes);
- excess expected mean crash frequency with EB adjustments: the excess is the difference between the expected crash mean frequency at the site and the predicted mean crash frequency from a reference SPF (the most dangerous sites show the highest excesses).

It is evident that the choice for one specific performance indicator depends both on the available data and the type of analysis. In fact, for example, if the focus is on a specific crash type, metrics based on crash types can be used. Moreover, if no traffic volumes are available, several metrics cannot be used.

The following advantages and disadvantages of the listed metrics can be highlighted (referred to the general metrics)[2]:

- crash frequency, crash rate, EPDO mean crash frequency and relative severity index are simple and immediately applicable indicators (the crash frequency is the simplest possible), but they do not take into account the RTM bias;
- even if, among the simplest metrics, the crash rate accounts for traffic, it may show very high values for low traffic volumes and few crashes (crash rates of sites with very different volumes are hardly comparable);
- metrics obtained by means of the EB method can account for the RTM bias and so they should be preferred, but they are typically based on SPFs, which may be not available (or for which local calibrations are not available);

[5] Kononov J. (2002), "Use of Direct Diagnostic and Pattern recognition Methodologies in Identifying Locations with Potential For Accident reductions", *Transportation research record*, 1784(1), 153-158.

[6] Midwest Research Institute (2002), *White paper for Module 1-Network screening*, Federal Highway Administration. U.S. Department of Transportation, Available on www.safetyanalyst.org/whitepapers.

- some methods (e.g., the LOSS) can at least account for the variance of data.

The application of the metrics for screening purposes results in rankings. High-ranked sites are those which mostly need a reduction in the frequency/severity of crashes, and thus sites for which countermeasures may be beneficial.

Moreover, the HSM suggests to apply different performance measures to the same set of sites, to reveal if the same sites are present in the high-rank positions (those requiring particular attention) or in the low-rank positions (those requiring less attention).

8.3 How to prioritise projects and interventions across the analysed segments and intersections

- The Guidelines use the Benefit-Cost Analysis and the Benefit/Cost Ratio;
- the HSM provides additional possibilities depending on the different objective: the BCR method may be someway limited. In fact, while the BCR provides an evident result referred to the single project alternative, which is useful for decision makers (viable alternative in case of BCR>1 or alternatively not viable), it does not make comparisons between alternatives or does not consider budget constraints.

8.4 The concern of identifying alternatives among the countermeasures and how prioritising them

- The Guidelines are generic;
- HSM provides practical criteria and methods.

8.4.1 Guidelines

The supervisor body makes the economic evaluation of the interventions to understand the most convenient and appropriate ones. This evaluation requires an interaction with the road owner/manager in order to achieve the stated purpose.

The main method used to assess the technical and economic convenience of an intervention on the road network is the cost-benefit analysis (CBA).

This analysis relies on all social benefits and costs coming from project realization. It is not an easy task defining the monetary value of things such as environmental or human life.

Aspects to be evaluated are:

- *impacts on road safety:* costs related to road crashes (statistical value of the human life, health costs, costs resulting from loss of productivity and reduced quality of life, costs from material damages, administrative costs);
- *impacts on mobility:* travel time cost, traffic congestion cost;
- *impacts on the environment:* noise and air pollution, visual intrusion, landscape impact.

The costs (C) and benefits (B) might be quantified, over the time period (n) which represents the economic lifetime of the investment. Only once the discount rate has been defined too, the cost-benefit judgement for implementing the intervention can be performed as based on:

- *net present value (NPV):* discounted difference between benefits and costs for a project without alternatives, the convenience exists when the sum of the benefits is higher or equal to the sum of the costs. In the case of several project alternatives the most convenient will be the one showing the highest NPV value;
- *the benefits/discounted costs ratio (RBCA):* the ratio between benefits and costs.

Therefore, the intervention cannot be considered convenient if NPV is negative or if the discounted B/C ratio (BCR) is less than 1.

8.4.2 HSM

CMFs can be used to estimate the effect of the chosen countermeasures or set of alternative countermeasures for a given site, up to a large set of sites.

Hence, the economic evaluation process can be based on the conversion of the estimated reduced crashes (through the application of CMFs, and SPFs for base estimates) into monetary values, to be compared with costs of countermeasures. In other words, the reduction in crashes results in a monetary benefit to be compared with the costs of intervention.

Different methods can be used to assess which alternative countermeasures are the most beneficial for given sites and/or the less expensive. The alternative interventions can be sorted in ascending/descending order based on the following criteria:

- project costs;
- monetary value of project benefits;
- a decrease in the total crashes (or in the most severe crashes);
- Net Present Value (NPV), as introduced in the previous sub-section;
- Benefit-Cost Ratio (BCR), as introduced in the previous sub-section;
- Cost-Effectiveness index, which directly compares the costs of countermeasures with the estimated reduction in crashes.

The different design alternatives can be ranked according to one or more of the above listed criteria. These rankings can support the decision-makers in choosing the most appropriate countermeasure or set of countermeasures to be implemented.

8.5 Final project design problem

Neither the Guidelines nor the HSM give specific support on this topic.

8.6 Data availability problem

- The Guidelines suggest to use crash data only, if traffic volumes are not available (see table below).
- The HSM suggests to use different indicators (see 8.2.2) depending on the available data.

Tab. 8.5: Site Ranking list based only on crash data, without traffic flow data[1].

Metrics		*(a)*	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Homogeneous segment	*Length*	*Death rate/ Vehicle flow*	*Deaths frequency*	*Deaths*	*Injury rate/ Vehicle flow*	*Injury frequency*	*Injuries*	*Classification based on the metric (b)*	*Classification based on the metric (c)*
	Km	*N. deaths/ 10^6 veic.*km*	*N. deaths/ Km*	*N.*	*N. deaths/ 10^6 veic.*Km*	*N. injuries/ Km*	*N.*		
A	5	Unavailable	1/5	1	Unavailable	3/5	3	3	3
B	3	Unavailable	1/3	1	Unavailable	3/3	3	2	2
C	2	Unavailable	1/2	1	Unavailable	3/2	3	1	1

8.7 References

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9. The friction diagram method

This chapter provides the basis and the explanations of a new methodology developed for solving the road friction design issues: The Friction Diagram Method, FDM. This approach has been developed by the same research group that is publishing this book even if, at the moment, no simulations have been run yet to verify its reliability and its fit to real data in different contexts. The FDM has been only theoretically tested in different examples (see the rural example of application in the chapter 13), but not all the results have been satisfactory. Hence, this procedure is not compulsory in the road safety intervention protocol, and each engineer should be free to use it or not whithin the proposed framework.

Moreover, the FDM, if accurately tested, could be automatically implemented on board of Autonomous Vehicles, in order to provide detailed information about the real-time friction potential and friction used.

9.1 Road friction, friction capital and friction diagram

Generally, human comfort and safety perception rely on the absence of dangers detected by the memory and the five senses. Human beings usually tend to trust these feelings. Therefore, they base the interactions with the external environment on such feelings. The external environment provides several stimuli which need to be processed by the human mind. The number of stimuli that a user receive is not constant, the number changes according to the driver conditions. If the driver is at rest, the number of received stimuli depends on the variability of the environment. If the driver is in motion, the speed affects the number of received stimuli. In fact, the higher is the speed, the higher is the variability of the external environment perceived by the user. The situation changes also if the user is walking or is in a vehicle.

In the first case, the recognition and interpretation of the surrounding environment is deemed to happen thank to the five senses. In the second case, not all the senses are involved in the interpretation of the environment and the ones used may have a limited perception. The sight and hearing still define the distance and the position of the obstacles or dangers; the touch is inhibited by the presence of the vehicle, standing between the driver and the car, so it becomes impossible to assess the slip condition. Therefore, the drivers have never the clear and whole perception of the friction exhibited by the tyre while rolling on the road surface. Hence, the drivers attempt to estimate the friction in an indirect way through the sight and the acceleration variation in the internal organs.

The lack of friction becomes noticeable just in case of wet or snow road surface and the skidding risk perception is an “on-off” mechanism. In other words, the user drives not caring at all about the friction if he labels the road surface as safe after a visual analysis. In this situation, the driver ignores when the skidding condition could happen, even if the friction could fail any time without warnings.

Thus, the most advisable situation for the driver should be providing him/her with a real-time information about the friction capital in order to adapt the driving behaviour to new varied friction conditions. In this way, it would be possible to prevent the vehicle stability loss.

The aim of this chapter is providing a new method of friction analysis, after having studied in detail the phenomenon. The new method has been called Friction Diagram Method (FDM), which lets the engineer, in the design phase, know the *distance* from the friction limit for each road section.

Designing a road (new roads or adjustment of existing ones) consists in defining, in detail, all the characteristics of the road elements, in order to ensure their functionality for any condition and for the whole infrastructure lifetime.

Some engineering standards have introduced the "Limit State", i.e. the limit after which a generic structure no longer meets the designed and imposed requirements. This definition implies that the safety of an element depends on the resistance and stress states. All the possible combinations of forces acting on the element are calculated, for the stress state. All the possible combinations of resistance factors which could affect the resistance of the element are calculated for the resistance state.

Nevertheless, even if many studies have confirmed that road safety is ensured by the combinations and interactions of "stresses" and "resistances" (human, vehicle, road, traffic, external environment), the road infrastructure standards do not widely include variability neither in the design phase nor in the check phases. The FDM method is a possible attempt to design a road considering the influences of the human-vehicles-traffic-environmental condition interactions on stresses and resistances.

These interactions are possible only thanks to the friction. In fact, each choice taken by the driver while driving, based on his/her perceptions, becomes an actual manoeuvre thanks to the friction effect. Although, the friction effect, in turn, depends on the road, on the vehicle efficiency and environmental conditions.

The Friction capital is the main baseline idea of the FDM. Likewise, a concrete or steel specimen could resist until a maximum compression or traction force, the same happens with the road. A road could resist at most until a certain tangential stress transmitted by the tyre of a vehicle under given boundary conditions; this maximum road resistance is the Friction Capital. The friction capital varies along the road layout with the road geometry, the road surface conditions and the tyre condition.

When a vehicle travels on the road, it uses only a part of this available capital and the less friction capital is used, the safer is the travel. The term *Distance* indicates the percentage difference between the value of the Available Friction Capital (Friction Potential) and the value of the Used Friction Capital (Friction Demand). Thus, the Friction Demand is the distance from the limit condition where the Used Friction Capital is the 100% of the Friction Potential.

The FDM quantifies this *distance* from the limit condition in all the road sections with respect to all the variables which affect the friction value: environment, road surface conditions, type of manoeuvre, vehicle characteristics, and so on. The FDM indeed enables the engineer to make friction checks along the whole road layout thanks to the diagram.

9.2 Critical analysis of a well-established approach for the friction problem: the III Safety Criterion of Lamm

Lamm et al. (1999)[1] introduced the 3rd Safety Criterion. The cited study analyses the influence of the road surface, vehicle performance and user behaviour on the friction conditions. It also provides a ranking about the safety degree of a road segment (good-fair-poor).

Lamm et al. (1999)[1] define the road consistency through a comparison between the friction that the road can provide to the vehicle, and the friction that the vehicle requires from the road. Therefore, Lamm gives a criterion for judging the safety of a road section based on the comparison between the side friction factor assumed "f_{RA}" and the side friction factor demanded "f_{RD}"[1,2,3].

The first coefficient "f_{RA}" is the maximum coefficient of friction in the cross direction that the road can provide to a vehicle under the design conditions, and it is defined by the following equation:

$$f_{RA} = n * 0.925 * f_t \qquad \text{(Eq. 9-1)}$$

1 Lamm R., Psarianos B., Mailaender T. R. (1999), *Highway Design and Traffic Safety Engineering Handbook*, McGraw-Hill, New York, USA.

2 Lamm R., Psarianos B., Cafiso S. (2002), "Safety evaluation process for two-lane rural roads", *Transportation Research Record: Journal of the Transportation Research Board*, 1796(1), 51-59.

3 Lamm R., Psarianos B., Choueiri E. M., Solilmezoglou G. (1998), "A practical safety approach to highway geometric design International studies: Germany, Greece, Lebanon and USA", In *International Symposium on Highway Geometric Design*, 9, 1-9.

Where:
- $f_t = 0.59 - 4.85 * 10^{-3} * S_D + 1.51 * 10^{-5} * S_D^2$

f_t is the longitudinal design coefficient of friction, varying with the design speed, S_D;
- 0.925 represents a reduction factor that corresponds to tyre-specific influences, a tyre expresses higher skid resistances in the longitudinal direction than in the cross direction;
- n is the utilization ratio of side friction. Thanks to n it is possible to define the maximum permissible tangential friction factor. n=1 is the maximum permissible longitudinal friction factor. For road segments in flat terrains the n value is around 0.45, for mountainous or rolling road segments the value is 0.40;
- S_D is the design speed.

The second coefficient "f_{RD}" (demanded side fiction factor) is the coefficient of friction that a vehicle requires when it runs a curve at the constant speed "S_{85}" which is the operating speed. In this calculation, the vehicle is modelled and assumed as a material point. The equation is the following:

$$f_{RD} = \frac{{S_{85}}^2}{127 * R_C} - e \quad \text{(Eq. 9-2)}$$

Where:
- S_{85} is the operating speed, i.e. the speed operated and not exceeded by the 85% of users;
- R_c is the radius of the curve;
- e is the cross slope of the road section.

The III criterion of Lamm defines the road consistency of a curved road section by comparing the factor for the assumed side friction (f_{RA}) and the factor for the demanded side friction (f_{RD}). The range values for each level of road consistency are shown in the table 9.1.

Tab. 9.1: III Safety Criterion of Lamm (based on Lamm et al., 2002[2]).

III Safety Criterion of Lamm	*Road Consistency*	*Good*	*[0.01 ; ∞)*
		Fair	*[-0.04 ; 0.01)*
		Poor	*(-∞ ; -0.04]*
		Recommended Range	$f_{RA} \div f_{RD}$

Therefore, the III Safety Criterion of Lamm is a safety measure, calculated according to the following hypothesis:
- coefficients of friction are exclusively influenced by the speed, the road cross slope and the curve radius;
- the vehicle speed is constant and equal to the operating speed;
- the vehicle can be modelled as a mass material point; therefore, the geometry of the vehicle has not any influence on the redistribution of vehicle weight on each single axis;
- the topography factor "n" completely describes all the possible longitudinal actions

According to these hypotheses, the applicability of the Lamm criterion is limited to the curves and any indication for other road geometric elements is not relevant. Moreover, this criterion neglects some fundamental mechanisms for the friction occurrence such as the rolling motion physics of the tyres, the influence of road geometry on the redistribution of the vehicle weight on the road pavement, road surface and environmental conditions, forces acting on the vehicles while travelling.

All these mechanisms are fundamental, so they are briefly explained in the following paragraphs.

9.3 Rolling motion physics of the tyres

A brief description of the classical modelling of the friction phenomenon occurring between two bodies in contact (the starting point of the III Safety Criterion of Lamm) is crucial before describing the physics at the base of the rolling tyres mechanism[4].

[4] Zagati E., Zennaro R., Pasqualetto P. (1998), *L'assetto dell'autoveicolo,* Ed. Levrotto & Bella, Torino, Italy.

Let us consider a body on a flat surface. This body received a force, F_t parallel to the ground. This body does not move until a certain value of F_t is reached, $F_{t,Lim}$. When $F_{t,Lim}$ is applied to the body, it starts to slide on the surface. This process implies that the surface is able to react with a force with a normal component, R_n and a transversal one, A. A is able to contrast the action of F_t until the maximum value of $F_{t,Lim} = A_{lim.}$

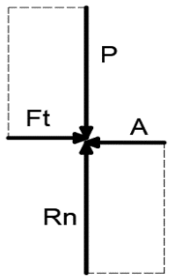

Fig. 9.1: Friction force on a surface.

The friction is:

$$A_{lim} = f_a \times R_n \qquad \text{(Eq. 9-3)}$$

Where:

- f_a is the coefficient of friction depending on the materials and on the conditions of the two surfaces in contact;
- R_n is the perpendicular component of the reaction forces on the flat surface.

The friction between two surfaces in contact is caused essentially by the characteristics of the two surfaces and by the force which acts "pushing" one body to the other. When the applied force F_t is parallel to the ground, and this is horizontal, the reaction R_n is equivalent to the weight W of the body.

The physics of the rolling motion of a tyre on the road surface is much more difficult, for a basic reason: the tyre motion is composed by two different mechanism, the rolling motion around the wheel axle and the translation motion of the axle in parallel to the rolling surface. The driving wheel of a vehicle can provide a traction force which moves and controls the vehicle itself; instead the driven wheel cannot do it. The torque moment "M" generated by the engine, acts on the wheel axle making the wheel spinning around the axis of rotation O'. The torque moment "M" acts in a reference plan perpendicular to the one where the axis rotation lays.

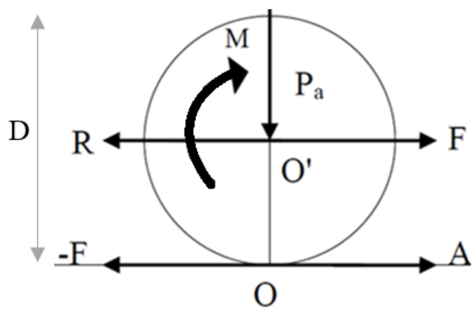

Fig. 9.2: Forces acting on the wheel.

- O' = wheel rotation centre;
- O = ideal contact point between the tyre and the road surface;
- M = |F| ·D/2 = torque moment;
- F = traction force applied in O'
- R = vectorial sum of longitudinal and shear forces acting on the running vehicle;
- A = friction force;
- P_a = adherent weight placed on the wheel (it is a rate of the vehicle weight insisting on the rolling mechanism perpendicular to the road surface).

When the vehicle starts moving, the torque M will go from 0 to the final value M_1, while M is increasing the force F will vary too, between 0 and $|F_1|$. These variations let the wheel passing the following stages:

- F will force the displacement of the point O' to the right, creating a passive force R (the sum of the actions acting on the travelling vehicle) in contrast with the displacement of O'.
  $0 < R < R_1$, R_1 is the final value reached by R: $R_{max.}$ If:
  - $F \leq R_1$, the point O' does not move (the vehicle is firm);
  - $F > R_1$, the point O' will move in the same direction of F (so the vehicle moves).
- The same F is applied in O. F will force O to go to the left, creating the pavement reaction to this displacement, A, with $0 < A < A_{LIM}$ (A_{LIM} is the maximum possible value reached by A). If:
  - $|F| \leq A_{LIM}$, the point O does not move (the friction occurs between tyre and road surface);
  - $|F| > A_{LIM}$, the point O will move in the same direction of F (so there is no developed friction in tyre-road surface contact).

The vehicle moves if $A_{LIM} \geq R_1$ and so if the maximum friction force provided by the road is higher or at least equal to the sum of transversal and longitudinal forces acting on the vehicle.

This is the relationship taken into account for the analysis of friction conditions along the road layout in all the possible horizontal/vertical circumstances:

- flat terrain tangent;
- uphill tangent;
- downhill tangent;
- flat terrain horizontal transition curve;
- uphill horizontal transition curve;
- downhill horizontal transition curve;
- flat terrain curve;
- uphill curve;
- downhill curve;
- tangent on a crest curve;
- tangent on a sag curve;
- curve on a crest curve;
- curve on a sag curve.

In order to calculate the friction for all of these circumstances, the engineer should know precisely all the factors affecting A_1 and R_1, as it shown in table 9.2.

Tab. 9.2: Factors affecting A_1 and R_1.

Factors affecting A_1	*Factors affecting R_1*
Adherent weight (it varies with the vehicle speed and road geometry)	Road geometry
Coefficient of friction (it varies with tyre wear, road surface conditions and wear, vehicle speed)	Speed
	Environmental conditions (e.g. wind variable in intensity and direction)
	Regime of flow (constant speed, acceleration, deceleration)

9.3.1. How the adherent weight varies with road geometry

The adherent weight is a rate of the vehicle weight acting on the rolling mechanism perpendicular to the road surface, as aforementioned. Thus, the adherent weight is not constant, but it is influenced by the cross slope of the road cross section[5]. A 4-driving wheel vehicle travelling on un uphill road segment characterized by a α longitudinal slope and null cross slope (figure 9.3) transfers to the road pavement a perpendicular force equal to $W\cos\alpha$. According to the hypothesis that the center of gravity is set on the center line of the vehicle, the adherent weight of the vehicle on a single wheel is calculated as follows:

$$W_a = \frac{W_{veic}\cos\alpha}{4} \qquad \text{(Eq. 9-4)}$$

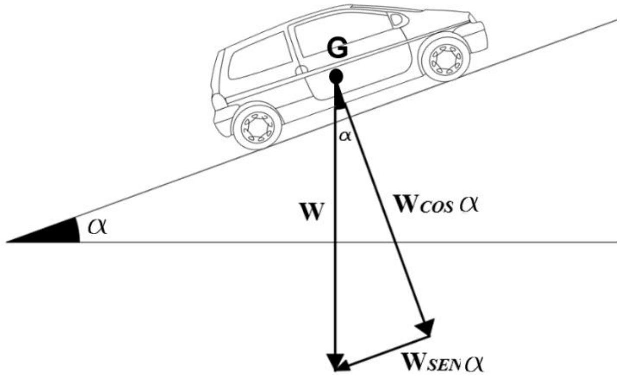

Fig. 9.3: Adherent weight of a vehicle on an uphill road.

A 4-driving wheel vehicle travelling on a road segment characterized by null longitudinal slope and Φ cross slope (figure 9.4) transfers to the road pavement a perpendicular force equal to $W\cos\Phi$. According to the hypothesis that the center of gravity is set on the center line of the vehicle, the adherent weight of the vehicle on a single wheel is calculated as follows:

$$W_a = \frac{W_{veic}\cos\Phi}{4} \qquad \text{(Eq. 9-5)}$$

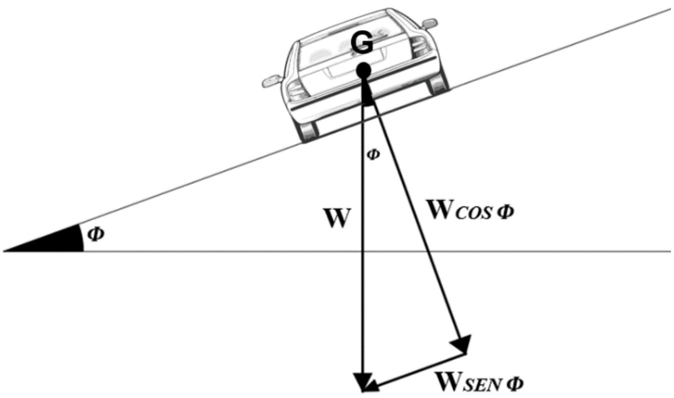

Fig. 9.4: Adherent weight of a vehicle on a road segment with Φ cross slope.

The adherent weight is indeed influenced by both the longitudinal slope, i_L, and the cross slope, i_C. The combination of these two slopes is the geodetic slope, i_G, calculated as follows:

$$i_G = \sqrt{i_L^2 + i_C^2} \qquad \text{(Eq. 9-6)}$$

The slope angle φ_G which the road creates with the horizon, is $\varphi_G = \mathrm{arctg}\,(i_G)$; so, the adherent weight Wa acting on the single wheel is expressed as:

[5] Colonna P., Berloco N., Intini P., Perruccio A, Ranieri V. (2015), "Development of a new method for analysing the road safety conditions related to friction", *Proceedings of the 6th international conference of automotive and transportation systems* (ICAT'15). Salerno, Italy, 27th-29th June.

$$W_a = \frac{W_{veic} \cos \varphi_G}{4} \quad \text{(Eq. 9-7)}$$

9.3.2 How the adherent weight varies with uphill or downhill roads

Until now, the influence of the pitching (or overturning) moment M’’ on the adherent weight has been neglected in the discussion. The pitching moment M’’ (Figure 9.5, 9.6) is due to the roadway slope, in fact the parallel component of the vehicle weight creates an overload on the downhill front axis and on the uphill rear axis.

The value of this overload could be calculated solving the static scheme in Figure 9.5 and 9.6:

- Downhill vehicle

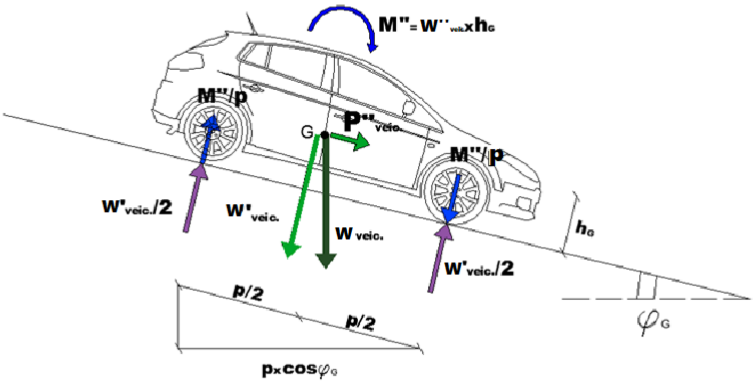

Fig. 9.5: Equivalent static scheme of a downhill vehicle (based on Colonna et al., 2015[5]).

The variables presented in Figure 9.5 are determined with the following equations:

$$W'_{veic,G} = W_{veic} \cos \varphi_G \quad \text{(Eq. 9-8)}$$

$$W''_{veic,G} = W_{veic} \sin \varphi_G \quad \text{(Eq. 9-9)}$$

$$M'' = W''_{veic,G} \cdot h_G \quad \text{(Eq. 9-10)}$$

Considering a front-wheel driving vehicle, the adherent weight on the driving wheels results:

$$W_{a,down} = \frac{1}{2} \cdot \left(\frac{W'_{veic,G}}{2} + \frac{M''}{p}\right) = \frac{1}{2} \cdot \left(\frac{W_{veic} \cos \varphi_G}{2} + \frac{W_{veic} \sin \varphi_G \cdot h_G}{p}\right) \quad \text{(Eq. 9-11)}$$

- Uphill vehicle

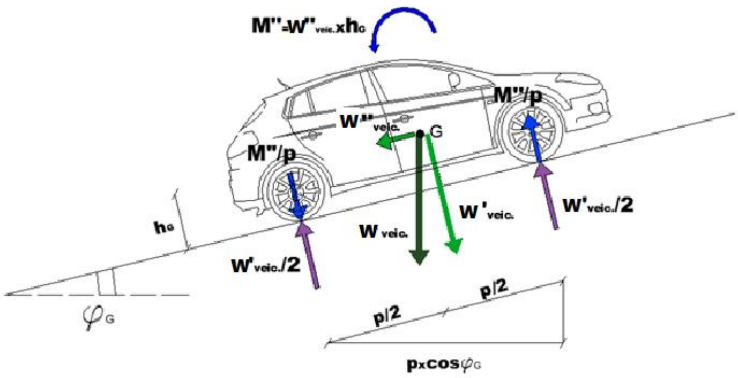

Fig. 9.6: Equivalent static scheme of an uphill vehicle (based on Colonna et al., 2015[5]).

The actions acting on the uphill vehicle are the same as those acting on the downhill vehicle, with the adherent weight reduced to:

$$\mathrm{W_{a,up}} = \frac{1}{2}\cdot\left(\frac{\mathrm{W'_{veic,G}}}{2} - \frac{\mathrm{M''}}{\mathrm{p}}\right) = \frac{1}{2}\cdot\left(\frac{\mathrm{W_{veic}}\cos\varphi_\mathrm{G}}{2} - \frac{\mathrm{W_{veic}}\sin\varphi_\mathrm{G}\cdot \mathrm{h_G}}{\mathrm{p}}\right) \qquad \text{(Eq. 9-12)}$$

9.3.3. How the adherent weight varies with crest and sag curves of the road

When a vehicle is on a crest curve, the driver and the passengers feel the well-known "weightlessness" effect, especially at high speed. The cause of this effect is to be attributed to an upward acceleration, due to the centrifugal force, inversely proportional to the radius of the vertical curve, acting on the vehicle running on the crest vertical curve. This force is opposed to the weight force and it causes a reduction of the adherent weight acting on the vehicle wheels. The force is expressed by the following equation:

$$F_c = \frac{m\cdot {V_{veic}}^2}{R_O} \qquad \text{(Eq. 9-13)}$$

The vehicle weight on the crest curve will be:

$$W_d = mg - \frac{m\,.V_{veic}^2}{R_O} \qquad \text{(Eq. 9-14)}$$

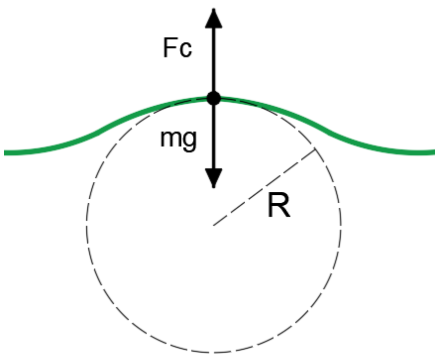

Fig. 9.7: Forces acting on a crest curve.

When the vehicle passes on a sag vertical curve, the effect is the opposite. In fact, the centrifugal force has the same direction and verse of the weight force, as shown in Figure 9.8:

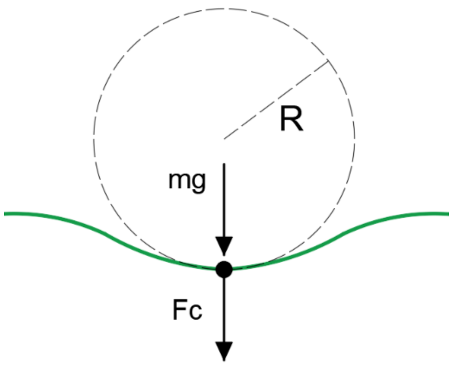

Fig. 9.8: Forces acting on a sag curve.

Thus, the vehicle weight transferred to the road is:

$$W_s = mg + \frac{mV_{veic}^2}{R_O}$$ (Eq. 9-15)

9.3.4 Environmental conditions and road surface conditions: the aquaplaning phenomenon

The friction coefficient "f_a" is strongly affected by the nature of the bodies in contact. In the road field, such coefficient is affected by the tyre conditions (wear, compounds, pattern, etc.) and by road surface[6] (wear, porosity, mix-design, etc.). Another factor which strongly affects the friction coefficient is the presence of water or dirtiness on the road surface, as well as the tyre pression and the vehicle speed. Figure 9.9 shows blatantly the variations of the friction coefficient in two circumstances: f_a expressed in function of the speed "V" while the depth "s" of the water layer varies, in the case of tyre with or without tread[6].

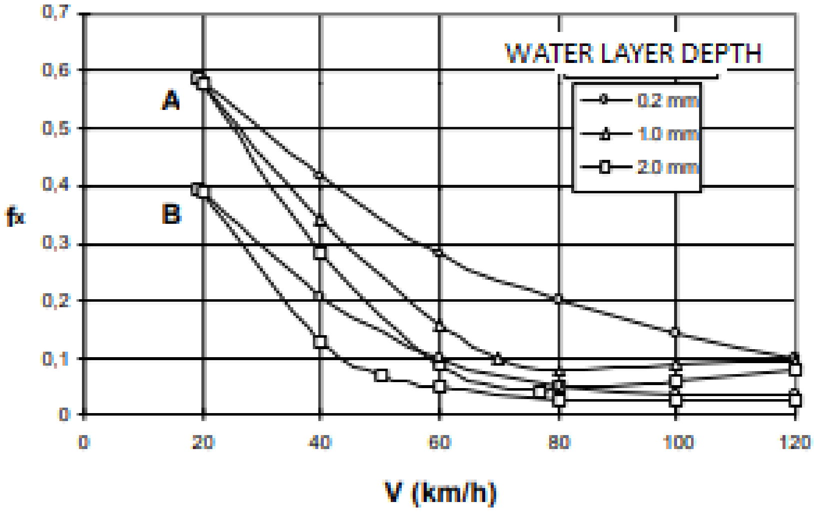

Fig. 9.9: Speed Diagram - Friction Coefficient. The curves show the friction coefficient versus speed if the tyre has the tread (A) or not (B), Canale et al. 1998[6].

The graphs show that f_a decreases when the speed and the depth of the water layer increase; there is a value of "V" at a given "s" that is the limit beyond which the tyre is raised by the water, so the aquaplaning occurs. In these conditions the value of the coefficient of friction is almost null and so the vehicle is without control.

9.3.5 Forces acting on a vehicle along the road layout

A vehicle in motion faces two kinds of resistances[7]:

- ordinary resistances;
- accidental resistances.

Ordinary resistances "R_0" occur to contrast the progressive movement in linear uniform motion of the vehicle (V_{veh}=constant) on a flat road. Accidental resistances, "R_e" are added to the ordinary resistances when the basic motion conditions vary, in curves, tunnels, longitudinal slopes. The total sum of the two resistances is as follows:

$$R = R_o + R_e = (R_1' + R_1'' + R_2) + (R_i + R_{curve} + R_G)$$ (Eq. 9-16)

Where:

- R_1' = rolling resistance due to the pin-bearing wheel friction (longitudinal action);

[6]Canale S., Leonardi S., Nicosia F. (1998), "Nuovi criteri progettuali per una politica di sicurezza stradale", *Proceedings of the 35th Road National Meeting (P.I.A.R.C.)*, Verona, Italia, May.

[7]Genta G. (1983), *Meccanica dell'autoveicolo*, Levrotto & Bella, Torino, Italy.

- R_1'' = rolling resistance due to the hysteresis of the tyres;
- R_2 = aerodynamic resistance composed by the cross action of wind gusts (cross action) and by the frontal wind gusts (longitudinal action);
- R_i = resistance due to longitudinal slope and cross slope of the road section;
- R_{curve} = resistance due to the presence of a curve (longitudinal action);
- R_G = inertial actions caused by acceleration and decelerations (longitudinal action).

This brief explanation of the factors which influence the vehicle motion blatantly show the necessity of dealing with the friction problem considering all the possible contributions.

9.4 The Friction Diagram Method (FDM)

The Friction Diagram Method FDM[8] is an instrument able to analyse the friction condition of a road layout considering simultaneously physical, dynamics and environmental factors, explained in the previous chapters.

This method provides an evaluation of the skid resistance on a road layout thanks to a synthetic parameter (F_{USED}). F_{USED}, once related to a specific Friction Diagram, allows to detect by an easy sight check the presence of high-risk road segment in terms of skidding.

The Friction Diagram could be built on a longitudinal profile of the road, drawing section by section the interpolating line of the F_{USED}.

The main ideas introduced by the FDM are essentially three:

- Friction Potential "F_P"
- Friction Demand "F_D"
- Friction Used "F_{USED}".

9.4.1 The friction potential

The idea of the Friction Potential is the same of the Friction Capital, which has been described above. The Friction Potential is the highest value of the Friction force that the road could provide, and it varies along the road layout with the adherent weight and the coefficient of friction. The equation to calculate it is provided below:

$$|F_P| = W_{a(g,s)} \times f_{a(c,s)} \; [N] \qquad \text{(Eq. 9-17)}$$

Where:

- $W_a(g,s)$ = adherent weight acting on drive wheels; it varies with the speed "s" and with the road geometry "g";
- $f_a(c,s)$ = friction coefficient in a generic direction; it varies with the speed "s" and with the road conditions "c".

9.4.2 The friction demand

The Friction Demand, already introduced in this chapter, is the Friction force that a vehicle requests to the road. It is equal to the sum of all the listed resistances (longitudinal and cross). It is possible to calculate it through the following equation:

$$F_D = \sqrt{L^2 + C^2} \qquad \text{(Eq. 9-18)}$$

Where:

[8] Colonna P., Berloco N., Intini P., Perruccio A., Ranieri V. (2016), "Evaluating the skidding risk of a road layout for all types of vehicle", *Transportation Research Record: Journal of the Transportation Research Board,* 2591(1), 94-102.

- L is the sum of the longitudinal forces: rolling resistance, aerodynamic resistance, slope resistance, curve resistance, wind resistance;
- C is the sum of the cross forces: centrifugal force, cross component of the weight force, cross action of wind gusts.

9.4.3 The friction used

The Friction Used is the synthetic parameter which enables to verify the friction conditions of the travelling vehicle. It is the percentage of friction that the vehicle is using compared to the maximum that the road can provide. In other words, F_{USED} represents the portion of "Friction Capital" that the vehicle consumes in certain conditions, therefore F_{USED} provides a measure of the distance from the limit condition in which $F_P=F_D$ (the friction potential is equal to the friction demand). The equation to calculate the friction used is:

$$F_{USED} = \frac{F_D}{F_P}\ x\ 100\ [\%] \qquad \text{(Eq. 9-19)}$$

Safety conditions are defined by the distance of the F_{USED} from its limit value which is 100%. So, in very safe situations F_{USED} will have a low value; in situations in which the skidding risk is high F_{USED} will have a value near 100%. Whereas, in situations in which vehicle stability is not ensured, F_{USED} will have a value greater than its limit.

9.4.4 FDM implementation steps

The steps useful to calculate the F_{USED} value for each section of the road layout and so to design the Friction Diagram are provided below:

- Geometry and motion analysis:
  - Cross slope of the road section, Φ;
  - longitudinal slope of the road, α;
  - Radius of curvature of vertical curves;
  - Radius of curvature of horizontal curves;
  - Parameter A and length s of the horizontal transitional curve;
  - Speed, acceleration and deceleration of the vehicle;
  - Determination of the friction coefficient fa, as a function of road condtions,c, and speed, s;
- Friction Potential Determination;
- Determination of the sum of longitudinal forces acting on the vehicle, L;

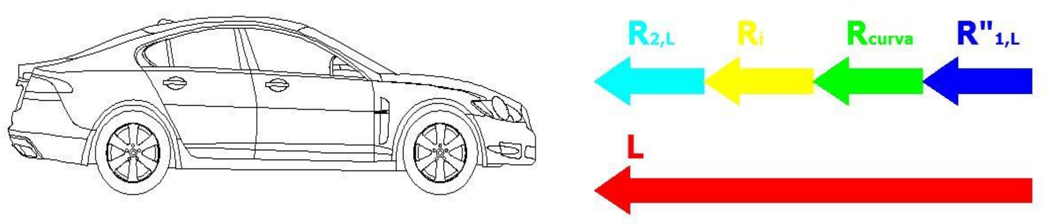

Fig. 9.10: Sum of longitudinal forces.

- Determination of the sum of cross forces acting on the vehicle, C;

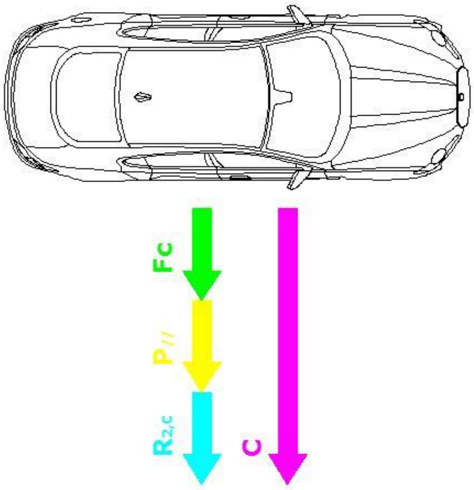

Fig. 9.11: Sum of cross forces.

- Determination of the sum of longitudinal and cross forces, R, which has the same intensity and direction of the Friction Demand, but it is in contrast with the Friction Demand, as shown in the figure below.

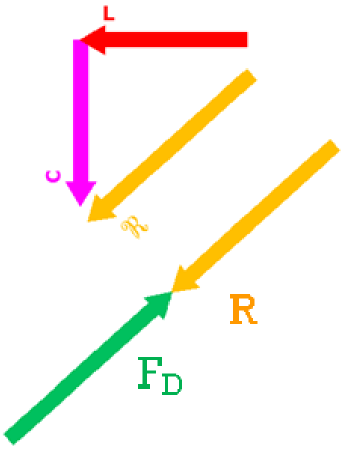

Fig. 9.12: Friction Demand opposing to the sum of all the forces, R, acting on the vehicle.

- Determination of the F_{USED} value through the comparison between F_D and F_P.

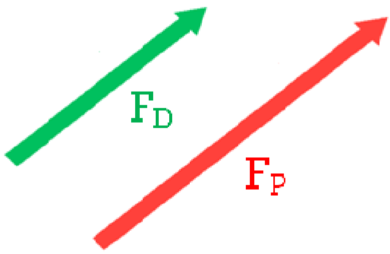

Fig. 9.13: Percentage comparison between FD and FP.

The FDM has been implemented in a spreadsheet which is applied quickly if there is the availability of data about the road layout and the vehicle, as shown in the table 9.3:

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Tab. 9.3: *Road layout data and vehicle data useful for the implementation of the FDM.*

Road layout data	*Vehicle Data*
Friction coefficient	Longitudinal (p_L) and cross pitch (p_C)
Geometry of the road section and vehicle travelling direction	Longitudinal section (S_L) and cross section (S_C)
Vehicle speed, acceleration, deceleration, wind speed and direction	Longitudinal shape factor $C(N)_L$) and cross shape factor ($C(N)_C$), Mass (m) Height of the centre of gravity (h_G)

9.5 An elementary example of the FDM application

This paragraph shows the example of one application of the Friction Diagram Method using both the spreadsheet and the diagram. The applicative case regards a road segment (from the km 6+000 to the km 8+000) belonging to the SP239, in the Metropolitan area of Bari as shown in the following figures (more detailed analysis and calculation are provided in the Chapter 13).

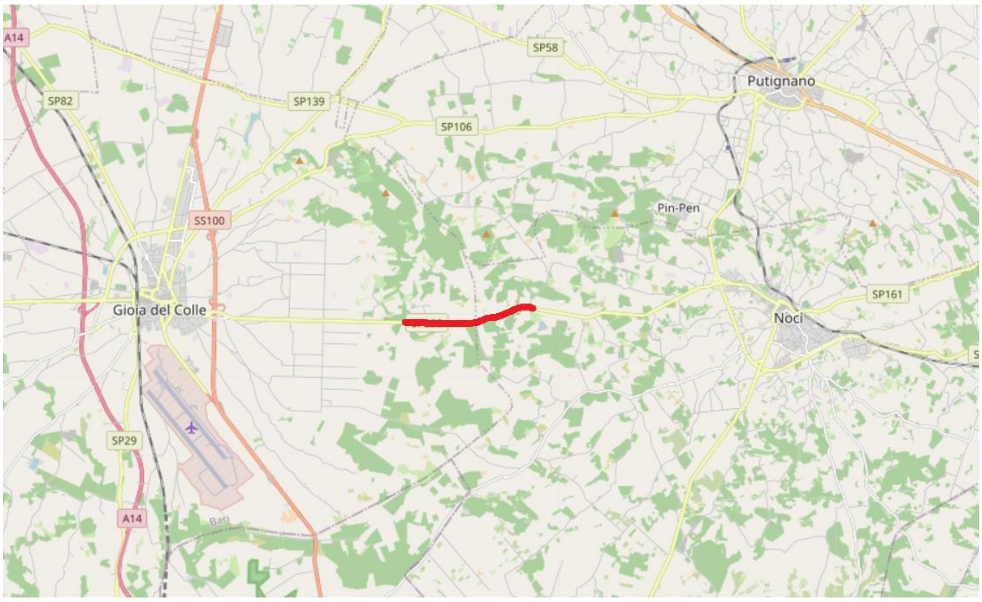

Fig. 9.14: Aerial photo of the analysed road segment in red (Open Street Map photo source).

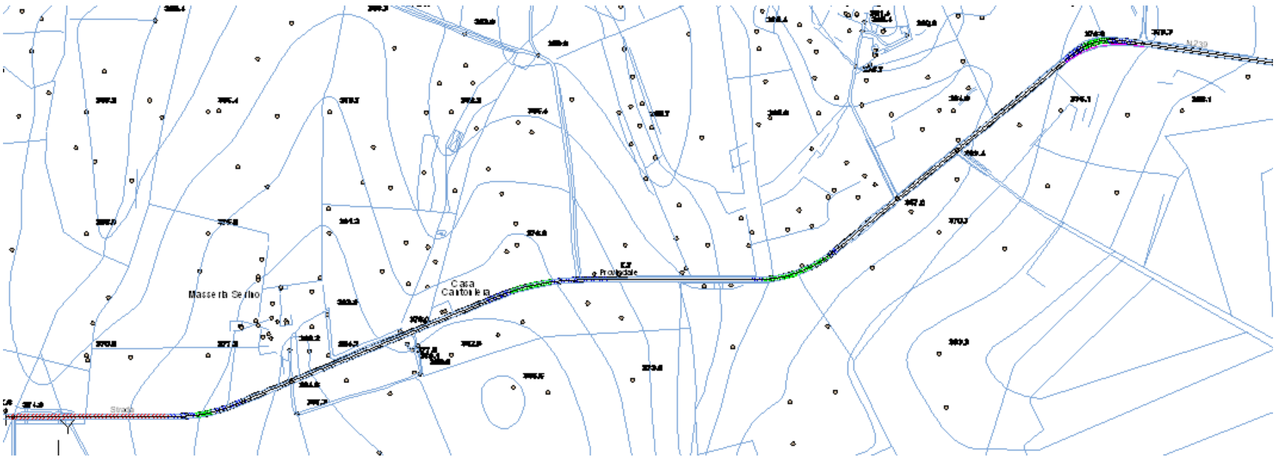

Fig. 9.15: Horizontal alignment on the regional technical map of the analysed road segment (see Chapter 13).

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The diagram about F_{USED}, shown below, was obtained using a “Fiat Bravo” model, a mid-sized sedan car (whose characteristics are reported in table 9.4) travelling on the SP239 road segment at a speed verified by the speed diagram. The speed diagram was calculated according to the hypothesis of wet road surface, according to the Italian road standards (D.M. 6792/01)[9].

Tab. 9.4: Fiat Bravo characteristics.

$C(N)_L$	$C(N)_C$	h_G [m]	$p_{,long}$ [m]	$p_{,Cross}$ [m]	m [Kg]	S_L [mq]	S_C [mq]
0.50	1.15	0.60	2.60	1.80	1320.00	3.12	2.16

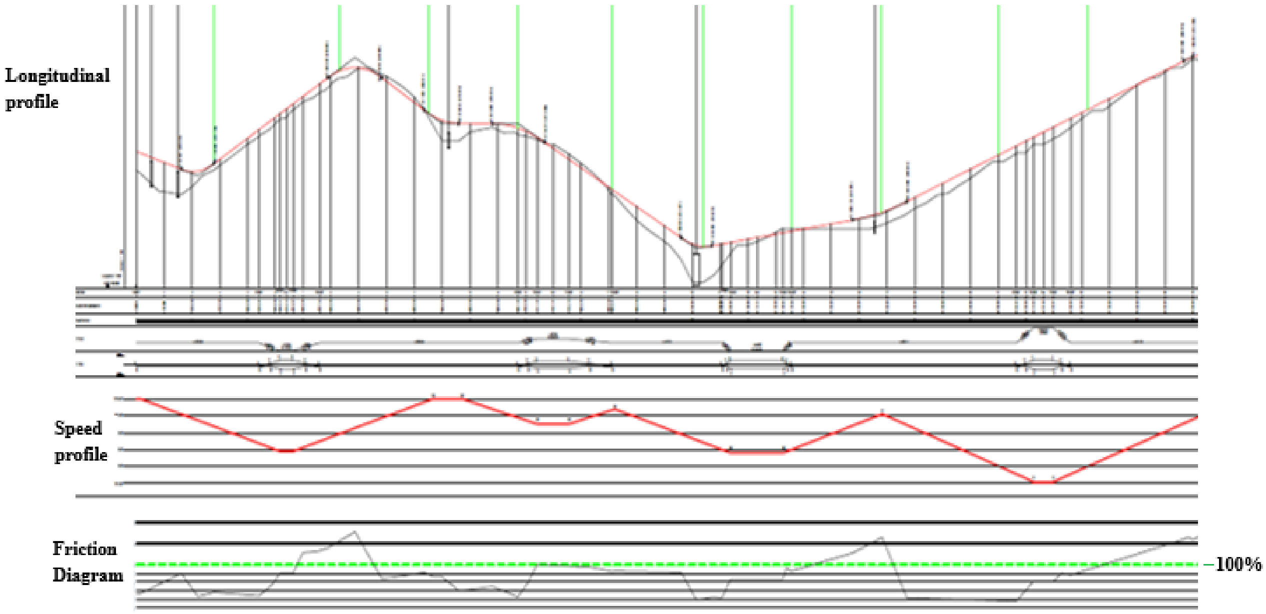

Fig. 9.16: Friction Diagram of the road segment belonging to the SP239, from the km 6+000 to the km 8+000.

The graph shows that in some segments (from 270 m to 420 m; from 1200 m to 1350 m; from 1720 m to 1900 m) the car could not travel at the speed given by the speed diagram without a skid resistance loss.

Therefore, the Friction Diagram could be also used to find out, and then to set, the adequate speed limits which could avoid the vehicle skidding on wet road surfaces[10].

9.6 The design critical vehicle and the complete application of the design procedure

The use of Friction Diagram Method requires knowledge of some forces acting on the vehicle. These forces are listed below:

- Weight force – depending on the mass of the vehicle;
- Inertial actions – depending on the mass of the vehicle and on the speed variation;
- Centrifugal forces – depending on the mass of the vehicle, the vehicle speed and on the radius of curvature of both horizontal and vertical curves;

[9] Italian Ministry of Transport and Infrastructures. Ministerial Decree n. 6792 of 5 November 2001: *Functional and Geometric Road Standards -Norme Funzionali e Geometriche per la Costruzione delle Strade*.

[10] Colonna P., Berloco N., Intini P., Ranieri V. (2017), “The method of the friction diagram: New developments and possible applications”, *Transport Infrastructure and Systems: Proceedings of the AIIT International Congress on Transport Infrastructure and Systems*, 309, Rome, Italy, 10th-12th April, CRC Press.

- Aerodynamic actions – depending on the wind speed and on the width of the longitudinal and cross vehicle surface.

Since these forces depend on the vehicle characteristics, the values of F_D and F_P, at fixed road conditions (other road conditions being equal), are different for each vehicle. Thereby, the F_{USED}, which is a function of the ratio between F_D and F_P, vary for each vehicle. Therefore, also the Friction Diagram is different for each vehicle, other road conditions being equal. Analysing the road safety based solely on the spreadsheet is therefore incorrect, since the Diagram is assessed just for an investigated vehicle. In that regard, the figure 9.17 shows the comparison among 5 different Friction Diagrams obtained by the analysis of 5 different kind of vehicles: city car, mid-sized sedan car, high performance sedan car, light commercial vehicle, 2-axis bus.

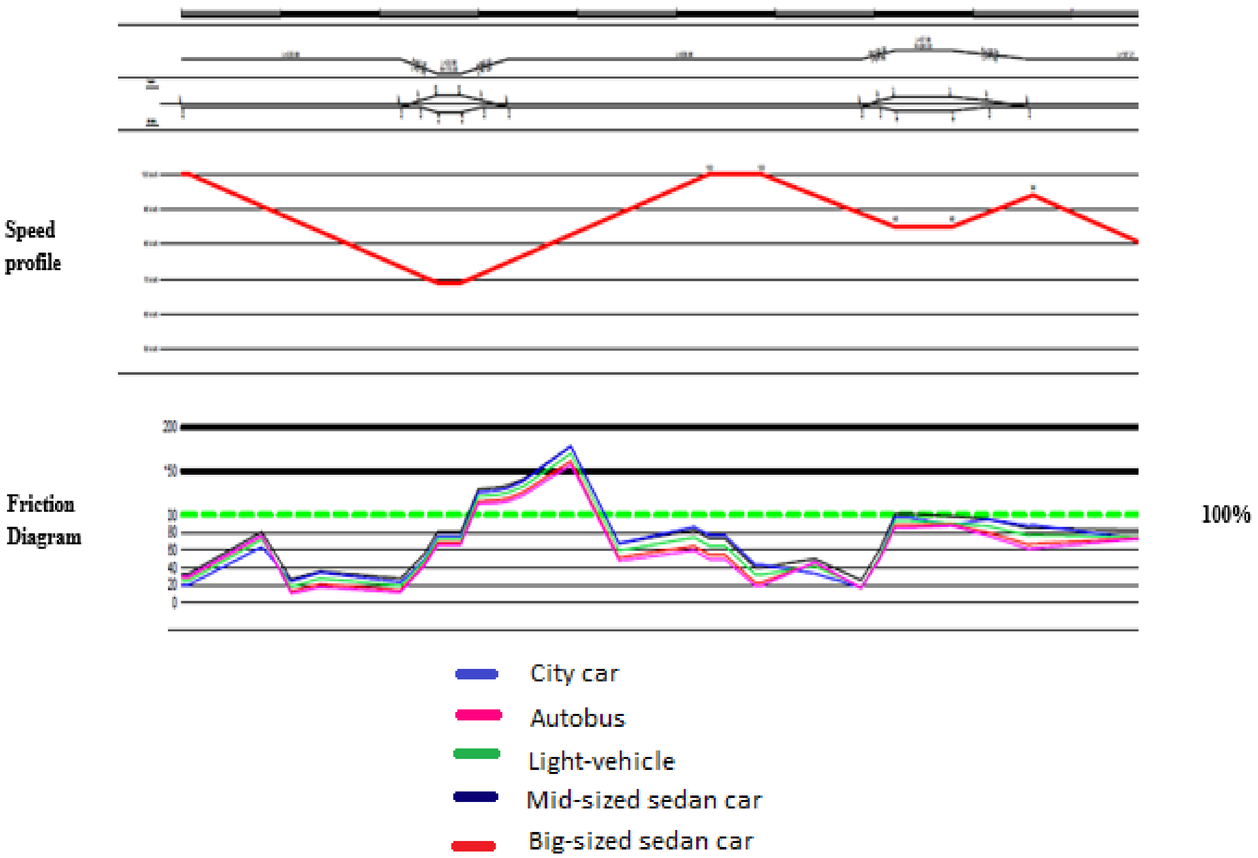

Fig. 9.17: Comparison among Friction Diagrams about the road segment of the SP239, from the km 6+000 to the km 8+000, for different vehicle types. Different colors stand for different vehicles.

The analysis of the F_{USED} diagrams shows that, in this case:

- there is not a kind of vehicle which always shows higher F_{USED} values than other kinds of vehicles;
- the highest detected F_{USED} values concerned the city car and the mid-sized sedan car;
- the lowest detected F_{USED} values concerned the city car and the heavy vehicle;
- the differences among the F_{USED} values is in the range from 0% to 35%.

Which is the vehicle to be used in order to draw the Friction Diagram without underestimating the level of the skidding risk? In other words, there is the necessity to determine the vehicle which maximizes the F_{USED} value.

For this reason, the FDM introduces the idea of a Design Critical Vehicle – DCV. This vehicle represents the starting point for the determination of the Friction Diagram of different road segments, each of them with its own specific geometry.

The main goal of defining the DCV is to put the practitioner in the conditions of drawing just one Friction Diagram to know the skid resistance of that road, independently from the kind of vehicle used for the analysis.

Through the study of numerous road segments and the consequent construction of numerous F_{USED} graphs as the vehicle types and motion conditions vary, it was observed that:

- the kind of vehicle, to which the maximum F_{USED} is linked, varies with the horizontal-vertical alignment conditions;
- on uphill and downhill roads, the F_{USED} value is strongly influenced by the type of traction system of the vehicle (front-wheels, rear-wheels or 4-wheels);
- the regime of flow (constant speed, acceleration, deceleration) deeply influences the F_{USED} values.

These considerations suggested to use a set of DCVs instead of a single DCV. Each vehicle of the set of DCVs, linked to a set of vehicle data, is critical for given combinations of road geometry and regime of flow.

Twenty different combinations were considered for all the possible horizontal-vertical alignment configurations, as shown in figure 9.18.

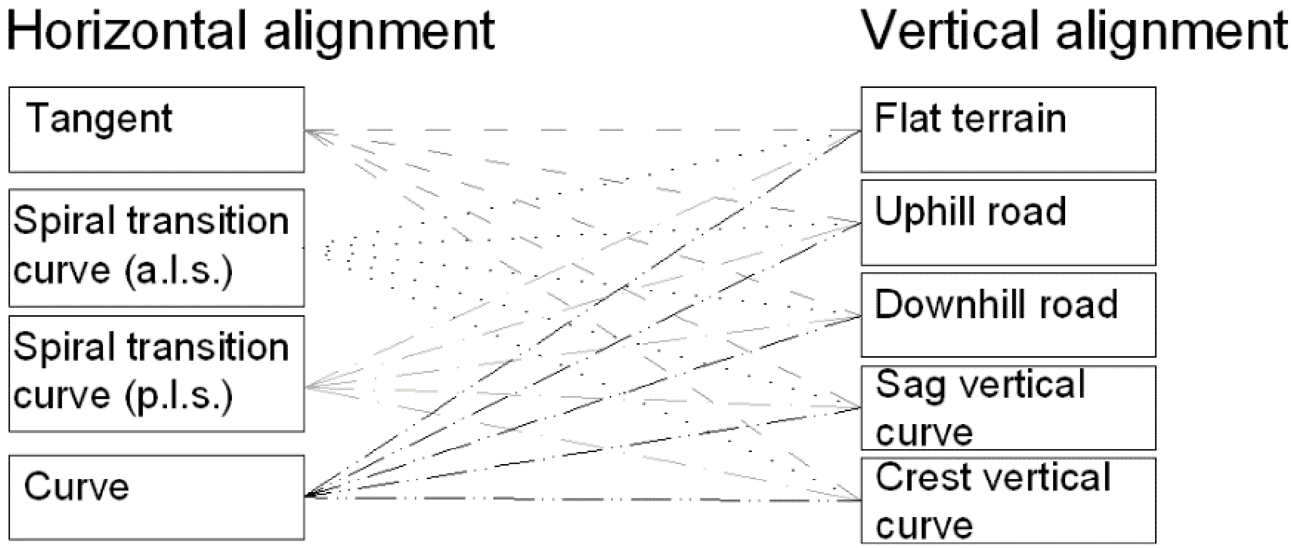

Fig. 9.18: Horizontal-vertical alignment combinations.

In figure 9.18, there is a distinction for the spiral transition curve of the horizontal alignment. In fact, in the spiral transition curve, there could be two different cross slope configurations: the first one has the cross slope of the road section oriented to the curve outside and so this part of the spiral curve is called ante level section (a.l.s.); the second one has the cross slope of the road section oriented to the curve inside and so this part of the spiral curve is called post-level section (p.l.s.). Such difference is important because the cross slope is crucial for the turning vehicle equilibirum at curves.

Instead, the motion regime was characterized by three configurations: constant speed, acceleration, deceleration.

The vehicle characteristics which affect the F_{USED} value for each of the previous explained combinations are the following: longitudinal (p_L [m]) and cross pitch (p_C [m]), longitudinal shape factor ($C(N)_L$) and cross shape factor ($C(N)_C$), mass (m [Kg]), height of the centre of gravity (h_G [m]), driving wheel (front-wheel/rear-wheel).

The study was conducted as follows:

- According to the size of usual vehicles, the variation ranges of the vehicle geometric characteristics are set:
  - 0.75 [m] < h_G < 1.5 [m]
  - 1.7 [m] < p_L < 6 [m]
  - 1.5 [m] < p_C < 2.5 [m]
- The following values were set to go in favour of safety:
  - m = 312.5 [Kg/m^3] (this is the mean weight per volume unit for each kind of vehicle)
  - $C(N)_L$, $C(N)_C$ = 1.15 (this is the minimum aerodynamic value).
- The combination to maximize F_{USED} was calculated for each combination of road geometry (20 configurations), motion regime (3 configurations) and driving wheel (2 configurations).

Tab. 9.5: DCVs data.

Critical Vehicle	$C(N)_L$	$C(N)_C$	h_G [m]	P_L[m]	P_C[m]	m [Kg]	Wheel Drive
I	1.15	1.15	1.5	6	2.5	7031.25	Front/ Rear
II	1.15	1.15	0.75	6	1.5	2109.38	Front/ Rear
III	1.15	1.15	1.5	1.7	1.5	1195.31	Front/ Rear
IV	1.15	1.15	0.75	1.7	1.5	597.66	Front/ Rear
V	1.15	1.15	0.75	6	2.5	3515.63	Front/ Rear
VI	1.15	1.15	1.5	6	1.5	4218.75	Front/ Rear

Among all the results, 6 recurring combinations were identified. Each of them represents the set of values of a DCV. The 6 identified DCVs and their characteristics are listed in table 9.5. table 9.6 presents the DCV to be linked to each of the possible combinations of road geometry, driving wheel and regime of flow.

Tab. 9.6: Identification of the DCV to be linked to each of the possible combinations of road geometry, driving wheel and regime of flow.

	Constant Speed	Acceleration	Braking	Wheel Drive
Flat terrain tangent	I	I	II	Front
Uphill tangent	III	III	IV	Front
Downhill tangent	III	I	II	Rear
Tangent in crest vertical curve	III	III	IV	Front
Tangent in sag vertical curve	I	V	II	Front
Flat terrain curve	I	VI	II	Front
Uphill curve	III	III	III	Front
Downhill curve	III	III	II	Rear
Curve in crest vertical curve	III	III	III	Front
Curve in sag vertical curve	V	V	II	Front
Flat terrain spiral transition curve (P|| Sfav)*	I	VI	II	Front
Flat terrain spiral transition curve (P|| Fav)*	I	I	II	Front
Uphill spiral transition curve (P|| Sfav)	III	III	III	Front
Uphill spiral transition curve (P|| Fav)	III	III	III	Front
Downhill spiral transition curve (P|| Sfav)	III	III	II	Rear
Downhill spiral transition curve (P|| Fav)	III	I	II	Rear
Spiral transition curve in crest vertical curve (P|| Sfav)	III	III	III	Front
Spiral transition curve in crest vertical curve (P|| Fav)	III	III	III	Front
Spiral transition curve in sag vertical curve (P||Sfav)	I	I	II	Front
Spiral transition curve in sag vertical curve (P||Fav)	I	I	II	Front

*(P || Fav and P|| Sfav stand, namely, for the favourable and adverse conditions of the same horizontal-vertical alignment).

Figure 9.19 provides a comparison between the F_{USED} diagram for the DCV (in red) and the F_{USED} diagram for the mid-sized sedan car (in black). The figure shows how the Friction Diagram drawn for a generic vehicle underestimates the F_{USED} value. The underestimation is between 5% and 30%. The DCV detects particularly dangerous conditions for the road segments included between: 270 m - 721 m and 1163 – 1700 m. In these segments, the F_{USED} value is over 100%, but this situation has not been detected previously by any of the tested usual vehicles.

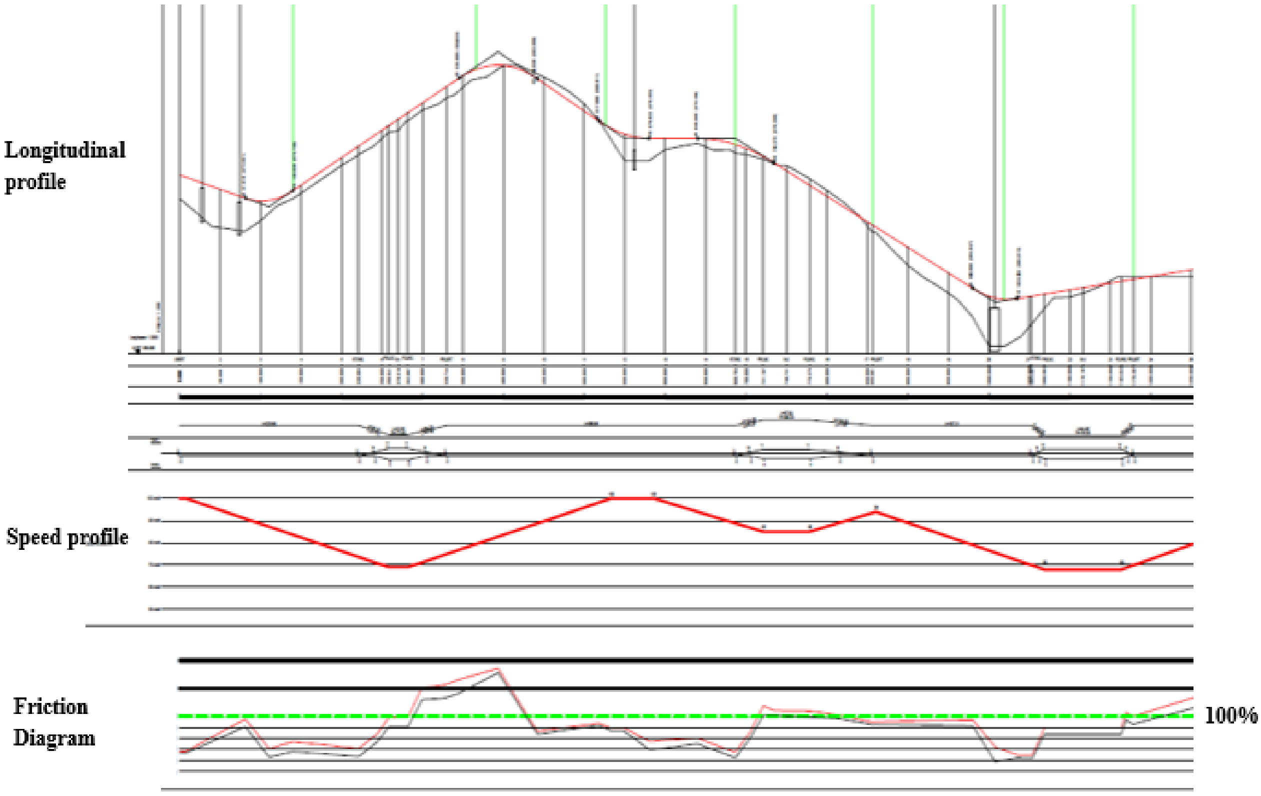

Fig. 9.19: Comparison between the F_{USED} diagram for the DCV (in red) and the F_{USED} diagram for the mid-sized sedan car (in black).

A threshold for the F_{USED} value could be set (e.g. 80%) and by means of the spreadsheet, the correspondent speed useful not to overcome the threshold may be determined. This calculated speed would be determined for the specific DCV associated to the road segment and to wet road surface conditions.

In this way, the new calculated speed diagram should be easily comparable with the speed limits on the analysed road segment, in order to easily prevent speeds leading to skid phenomena[10].

An application of this method could be a strong design tool together with the speed diagram, introduced by the Italian standards (DM 2001).

9.7 How could the friction be linked to future scenarios with autonomous vehicles

In a future scenario, where the majority of the circulating vehicles on roads will be driverless (or autonomous), the rural context may be deeply modified in order to allow Autonomous Vehicles (AVs) better performances. How could the infrastructure change with the introduction of AVs? Advanced Cruise Control (ACC), Antilock Braking System (ABS), lane guidance systems, Vehicle to Vehicle (V2V), Vehicle to Infrastructure (V2I) and Infrastructure to Vehicle (I2V) communication[11, 12] are all systems which should help the AVs with the comprehension of the surrounding world. Although, hacking attacks or software crashes could lead to uncertainties, so redundant systems and technologies are essential to overcome these possible concerns. While waiting that technological improvements will make the AVs an actual and safe reality in the road environment, it is possible to anticipate the situation understanding how the infrastructure concept and rules/standards could vary with AVs. Definitely, the materials of the road pavement could be different however ensuring the development of an adequate friction force.

[11]Colonna P., Intini P., Berloco N., Ranieri V. (2017), "Connecting rural road design to automated vehicles: The concept of safe speed to overcome human errors", In *International Conference on Applied Human Factors and Ergonomics,* Springer, Gewerbestr, Switzerland.

[12]Intini P., Berloco N., Colonna, P., Ranieri V. (2019), "Rethinking the main road d Automated Vehicle", *European Transport Issue 73,* Paper n° 3, 1-28.

Road design has always relied on physics and driver safety and comfort, but without drivers, how could the design change?

The following concerns the horizontal alignment:

- the maximum and minimum tangent lengths are linked to the human perception and comfort, so they could be avoided with AVs;
- the minimum curve radius is linked to speed, slope and available friction;
- the spiral transition curves respect physical requirements.

Hence, the horizontal alignment is mostly set to give a self-explaining road to the drivers, comfortable and easy[13].

The vertical alignment is limited by physical thresholds (water flow path, slopes, friction) and visibility and it has to be consistent with the horizontal alignment.

The human influence on visibility is fundamental, as the designed speed, V_d should suggest the correct safe speeds to be followed. It should be also harmonic with the road layout. The operating speed V_{85} (the operating speed not exceeded by the 85% of drivers) tests if the designed speed is correct or not (if V_{85} is higher or closer to V_d it means that V_d is substantially wrong). Posted speeds are set for safety purposes and they are very close to V_{85}.

Linked to the speed adopted by the drivers, there is a recommended friction coefficient in the design phase to ensure a safe travel for the vehicles. A minimum and acceptable value of the friction coefficient must be ensured even for AVs, since the friction coefficient does not depend on the drivers (except for the speed he/she chooses for the travel).

In the case of AVs, also, the Friction Potential, F_p, given to the vehicles by road, must be higher than the Friction Demand.

These forces depend on road geometry and materials, vehicle characteristics and speed. Hence, starting from these data, the F_{USED} for each section is determined.

Thinking about roads for AVs, even if the friction coefficient requirements are still essential, there would be many different aspects from current roads. Setting boundaries on the length of the tangent will be useless because the vehicle will be driverless. Instead, the boundaries on the minimum curve radius will still be useful because they affect friction and other physical properties occurring during the vehicle motion. However, the I2V communication should help the AV notifying the presence of the curve and the suggested speed to adopt in order to have the optimal friction and to keep high safety levels.

In this context, where the communications between vehicles and infrastructure is crucial, the way the central communication system operates is fundamental. The aim of the communication with vehicles is to provide vehicles with optimized information in order to rationalize the vehicles' route on the road. In this case, the V2V system may improve the efficiency of communication systems enhancing the capability of preventing traffic conditions and potential danger situations (crashes).

The transition curves in this scenario need to be set for ensuring the transition from one speed to another one. According to calculations about the speed difference recorded before and after the transition curve, the lengths of the spiral transitional curve could be set.

The roads should still ensure comfort for passengers, so the vertical alignment could be smoother with lower radius values than those required for a traditional condition. However; the vertical alignment must be always coordinated with the horizontal alignment.

The visibility problems will not need to be taken into account because the AVs could analyse the road with a 360° perspective, for long and short space ranges, thanks to sensors and cameras (long-range, mid-range and short-range radar; near-range and long-range camera) informing about obstacles and their current position. Hence, integrating the I2V communication with data coming from sensors constant update of the driving environment may be always ensured.

If visibility is negligible in the road design, the road will impact in a less severe way the environment. Knowing the traffic and the friction in real-time (thanks to GPS, torque measurements and accelerometers)[14,15,]

[13] Theeuwes J., Godthelp H. (1995), "Self-explaining roads", *Safety science*, 19(2-3), 217-225.

[14]Hahn J., Rajamani R., Alexander L. (2002), "GPS-based real-time identification of tire-road friction coefficient",*IEEE Transactions on Control Systems Technology*, 10(3), 331-343.

[15]Li L., Wang F., Zhou Q. (2006), "Integrated longitudinal and lateral tire/road friction modeling and monitoring for vehicle motion control", *IEEE Transactions on intelligent transportation systems*, 7(1), 1-19.

[16,17], the vehicle dynamics could be controlled (ABS) and the operating/posted speeds will lose their importance in favor of a better, safer and more dynamic speed, constantly updated for each section according to the variation of the external conditions. These assumptions suggest that the the central unit for the communication could receive info about geometry and traffic, in real-time, thanks to sensors installed at the roadsides. In this way, the traffic is controlled. The vehicle which enters each section, could gain info from the central unit thanks to the I2V communication and it may consequently update its speed considering an acceptable value of the estimated friction force for that specific section. On the other side, with the I2V communication, the vehicle could communicate its speed to the central unit, which compares all the three friction forces (potential, demand and used). As a response to the vehicle input, the central unit accepts or denies the speed value basing it on the evaluation of the friction force used. The vehicle could share info with other vehicles too, thanks to the V2V system. This is an iterative process, essential to ensure safety conditions even with AVs.

A schematic summary of how safe speeds could be governed and set on Autonomous Vehicle Network Roads is listed below, related to a rural curve section:

- a central unit may store basic default information on traffic and road geometry;
- sensors on the road infrastructure connected to the central unit will add information concerning current conditions (e.g. weather, water depth, etc.);
- a safe speed is determined by the central unit according to both default and actual conditions and shared with vehicles entering the section, through I2V connection;
- the vehicle entering the section will provide actual friction measurements on the road through its sensors, and communicate (V2I) these data to the central unit;
- the safe speed may be updated considering the actual friction measurements and shared with the following vehicles (I2V communication);
- the system will act as an iterative loop, considering also the potential redundancy provided by the V2V communication, having data shared between vehicles.

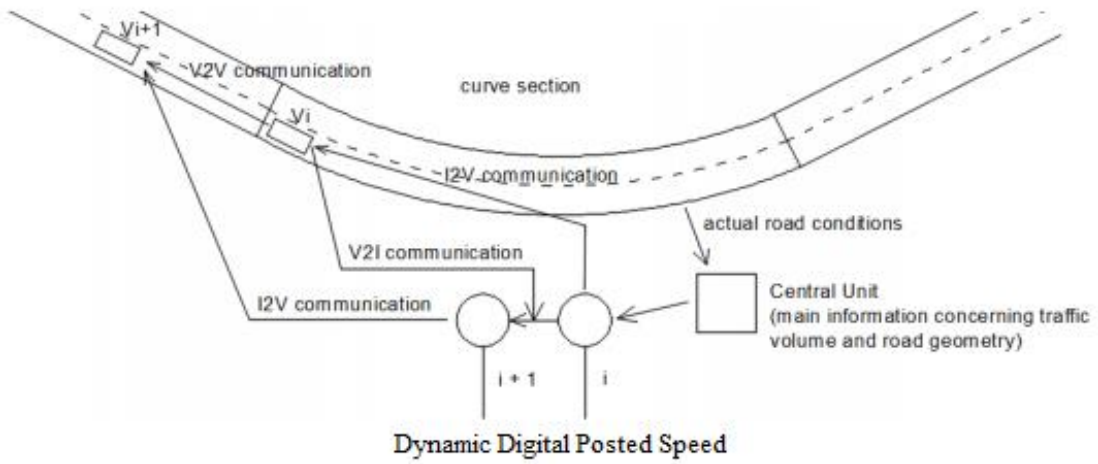

Fig. 9.20: The possible communication system among vehicles, sensors and units, to dynamically determine the posted speeds (Intini et al., 2019[12]).

This simple example assumes independency between road sections and the communications will be included within a limited area. However, vehicular and infrastructure data may be continuously shared among the network. The quantity and the distance of data sharing will depend on the technology advancement and the capacity to handle data, by simultaneously avoiding security issues.

A good improvement in the V2I communication as well as in the positioning system could reduce the time required for the AVs implementation in the current rural and suburban environment.

[16] Rajamani R., Phanomchoeng G., Piyabongkarn D., Lew J. Y. (2011), "Algorithms for real-time estimation of individual wheel tire-road friction coefficients", *IEEE/ASME Transactions on Mechatronics*, 17(6), 1183-1195.

[17]Choi M., Oh J. J., Choi S. B. (2013), "Linearized recursive least squares methods for real-time identification of tire–road friction coefficient", *IEEE Transactions on Vehicular Technology*, 62(7), 2906-2918.

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Hahn J., Rajamani R., Alexander L. (2002), "GPS-based real-time identification of tire-road friction coefficient",*IEEE Transactions on Control Systems Technology*, 10(3), 331-343.

Intini P., Colonna P., Berloco N., Ranieri V. (2019), "Rethinking the main road d Automated Vehicle", *European Transport,* Issue 73, Paper n° 3, 1-28.

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Lamm R., Psarianos B., Mailaender T. R. (1999), *Highway Design and Traffic Safety Engineering Handbook*, McGraw-Hill, New York, USA.

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Rajamani R., Phanomchoeng G., Piyabongkarn D., Lew J. Y. (2011), "Algorithms for real-time estimation of individual wheel tire-road friction coefficients", *IEEE/ASME Transactions on Mechatronics*, 17(6), 1183-1195.

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10. Level of Service of Safety (LOSS)

The relationship between average daily traffic (AADT) and the number of crashes may be complex and usually it is not linear. The relationship between AADT and crash frequency has been called Safety Performance Function (SPF), as widely discussed in the previous chapters (see also Hauer and Persaud, 1997[1], for a first introduction to this topic). The SPFs are functions that estimate the average crash frequency for a specific type of site (with some predefined baseline conditions) mainly as a function of the average annual daily traffic (AADT) and, in the case of road sections, of the length of the segment[2]. In case of intersections, traffic can be also differentiated according to the main and secondary intersecting roads.

The development of a SPF requires an intense crash data collection (e.g. five years of crashes were used for the development of a urban freeways SPF[3]) and the localisation of crashes on the network under investigation, to differentiate crashes occurred in intersections and segments (which are usually divided into homogeneous segments having constant geometric and traffic characteristics).

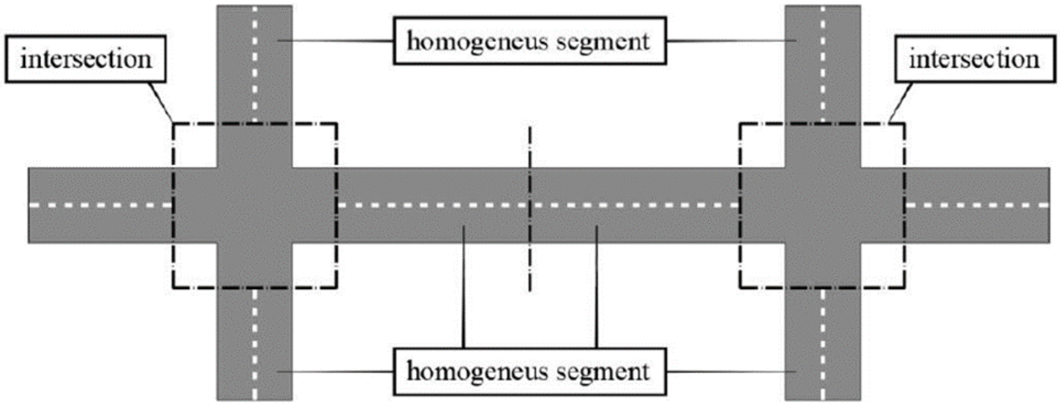

Fig. 10.1: Example of influence areas of segments and intersections.

The development of the SPF model is preparatory to explain the concept of the Levels of Service of Safety (LOSS)[4], firstly defined by Kononov and colleagues. The following paragraphs expose some concepts firstly introduced in the bibliography of these authors.

10.1 Definition of LOSS starting from the SPFs

The LOSS concept[4] is based on a qualitative/quantitative measure of the expected safety performances of a road segment. The level of safety assessed by the SPF represents the predicted number of crashes for a specific

[1] Hauer E., Persaud. B. (1997), *Safety Analysis of Roadway Geometric and Ancillary Features*, Transportation association of Canada.

[2] AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

[3] Kononov J., Bailey B., Allery B. (2008), "Exploratory Examination of the Functional Form of Safety Performance Functions of Urban Freeways", *Presentation at the 2008 TRB Annual Meeting*.

[4] Kononov J., Allery B. (2003), "Level of Service of Safety: Conceptual Blueprint and Analytical Framework", *Transportation Research Record: Journal of the Transportation Research Board*, 1840(1), 57-66.

AADT value. Therefore, the deviation from this value (±1.5σ) can be used to define four levels of service of safety (see figure below).

In particular, the four indentified areas will reveal if a specific site (based on its AADT value and observed mean crash frequency) is:

- highly safe with respect to the average predicted crash frequency from the SPF (site in the LOSS I area);
- safer than expected, below the SPF (site in the LOSS II area);
- less safe than expected, above the SPF (site in the LOSS III area), with some potential for safety improvement;
- highly unsafe with respect to the SPF (site in the LOSS IV area), with great potential for safety improvement.

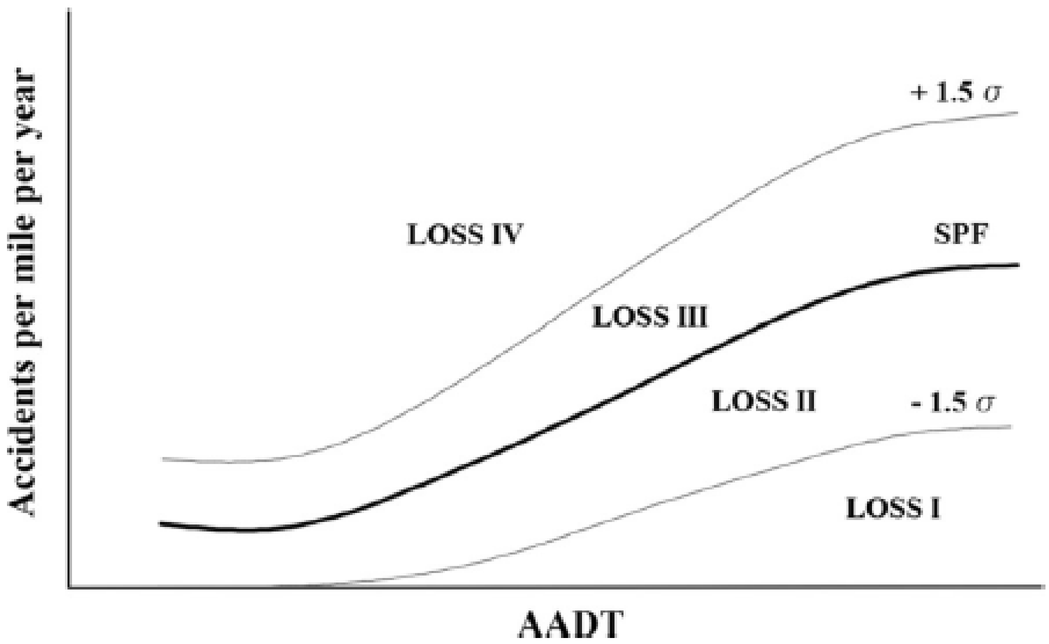

Fig. 10.2: The four Levels of Service of Safety (LOSS), based on Kononov and Allery (2003)[4].

In 2019, Kononov et al. (2019)[5] proposed an overhaul of the LOSS concept, suggesting some changes listed below:

- replacing the Observed Crash Frequency with the Expected Crash Frequency (using the Empirical-Bayesian EB method defined in Chapter 7), while evaluating the safety performance of a given site with respect to the SPF;
- defining boundaries between the different LOSS by using percentile values of the gamma distribution (for example 10% and 90%), rather than standard deviations.

Through these modifications, the following advantages are achieved (other conditions being equal):

- the RTM bias is corrected by using the EB method;
- data which are asymmetrically distributed around the SPF mean can be accounted, which was a limit of the previous criterion;
- the method can be more easily implemented by practitioners.

From the application of the LOSS method to different SPFs, the authors noted that a given road segment may have a different LOSS if it is calculated considering the crash frequency or the crash severity.

Furthermore, in general, the authors noted that while the AADT increases, the variability of the crash frequency per kilometre increases too, this is the reason why the LOSS I/II and III/IV dividing lines move away from the mean while sliding on the x-axis.

[5] Kononov J., Durso C., Lyon C., Allery B. K. (2019), "Level of Service of Safety Revisited", *Transportation research record,* 2514(1), 10-20.

However, the LOSS only describes how great the safety problem is, but the nature of the problem should be anyway determined through diagnostic analysis[4], to be conducted for the specific site under examination (see the final paragraph of this section).

10.2 Focus on the relationships between crash rates, traffic density and the number of lanes

In this sub-section, a focus on the relationships between crash rates, traffic density and the number of lanes, made by the same authors who developed the LOSS concept is reported[6,7,8] also related to the fact (previously highlighted) that SPFs may be not linear.

The crash frequency resulting from the SPF (see previous figure) is expressed in crashes per mile (or km) per year, but it can be easily converted into crash rates measured as crashes per million vehicle miles (km) travelled (million vehicle miles travelled VMT or million vehicle kilometres travelled VKT) as follows:

$$\frac{(\text{N. } crashes/km/year)\cdot 10^6}{\text{N. } vehicles\ per\ day \cdot \ 356\ \text{days}/year} = \text{N.}\, crashes/million\ VKT \qquad \text{(Eq. 10-1)}$$

In the following figure, the slope of the line which connects the origin with each point on the SPF can be considered as proportional to the crash rate above defined. The slope variation of the lines connecting the origin with the given points belonging to the SPF is then representative of such crash rate variation. Hence, the following graph highlights the crash rate variation as the AADT changes (given that the SPF is evidently not linear in this example).

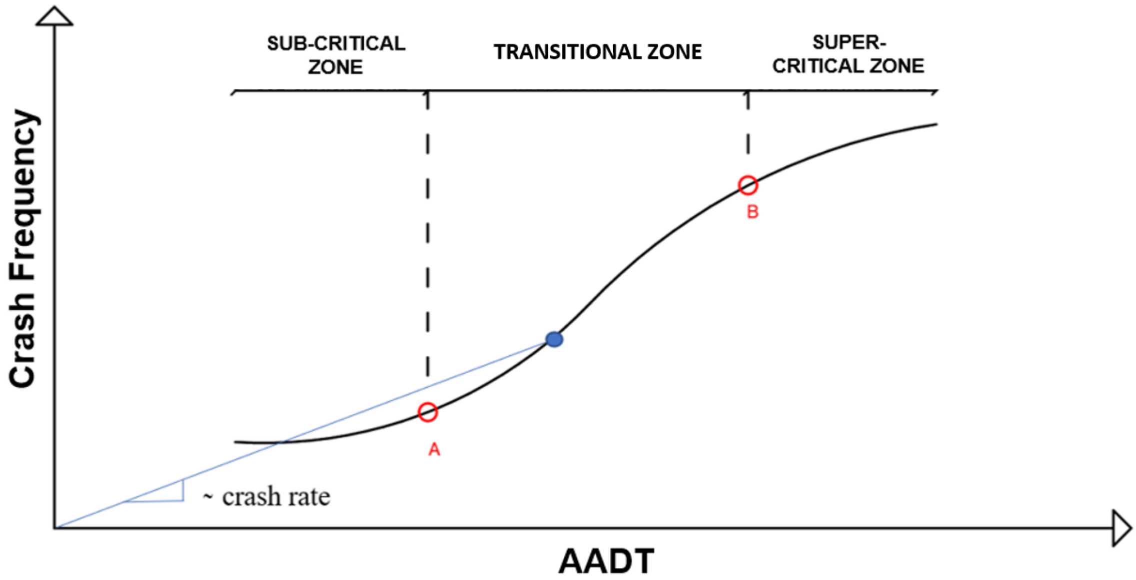

Fig. 10.3: Sub-critical, critical and super-critical zones (based on Kononov et al., 2011[6]).

Also, the authors elaborate[6] that it is possible to appreciate two critical points, A and B, for which the variation of the function gradient changes significantly.

The abscissa of the point A can be identified as a critical traffic density value, beyond which crashes increase faster by following the curve. The part of the SPF on the left of the critical density value is considered as a "subcritical zone", in which crashes increase slowly. The traffic density value after point B is a Super-Critical Density value. On the right of the point B, crashes slowly increase with the AADT, but the crash rate decreases.

The portion of the SPF included between the Critical and Super Critical Density values can be defined as a "transitional zone".

[6] Kononov J., Lyon C., Allery B. (2011), "Relation of Flow, Speed, and Density of Urban Freeways to Functional Form of a Safety Performance Function", *Transportation Research Record: Journal of the Transportation Research Board*, 2236(1), 11-19.

[7] Kononov J., Reeves D., Durso C., Allery B. K. (2012), "Relationship between freeway flow parameters and safety and its implication for adding lanes", *Transportation research record*, 2279(1), 118-123.

[8] Kononov J., Bailey B., Allery B. (2008), "Relationships Between Safety and Both Congestion and Number of Lanes on Urban Freeways", *Transportation Research Record: Journal of the Transportation Research Board*, 2083(1), 26-39.

However, from point A to point B, the following conditions should be applicable:

- AADT increases;
- traffic density increases;
- the operating speed decreases (even if the speeds remain similar in a first stage).

The crash probability should increase when the traffic density increases, and speeds are high. When the traffic density increases, considering typical freeway speeds, the road conditions becomes such that reacting for correcting a driving mistake may be hard. Hence, under similar speeds, the crash likelihood should be higher at higher densities. This concept is also explained graphically by figure 10.4.

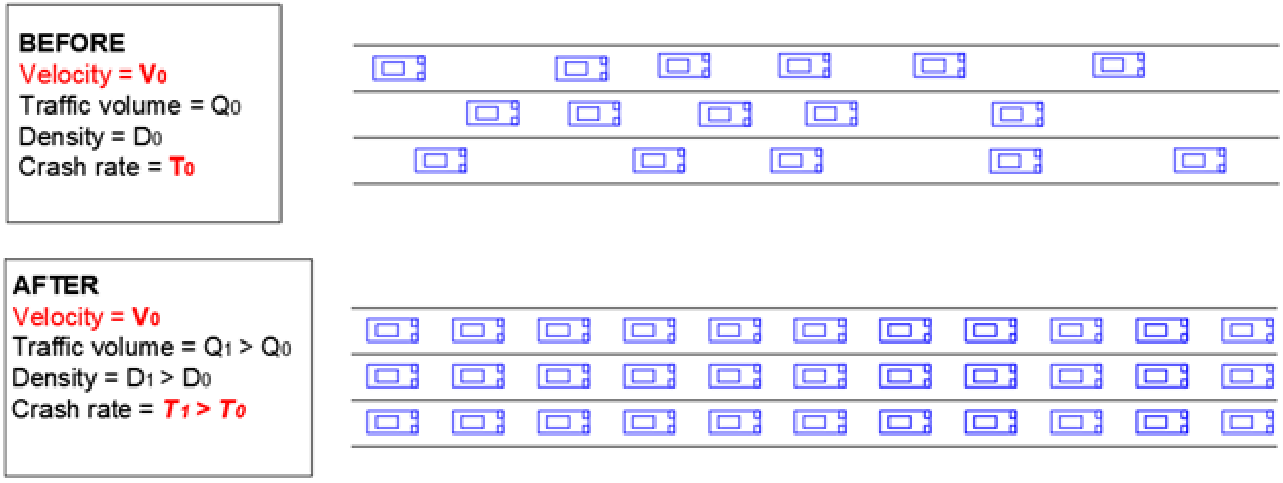

Fig. 10.4: The crash probability, for an equal speed, is higher at higher vehicle densities (Kononov et al., 2011[6]).

The integration of the concepts of the SPF and the LOS (Level of Service) allow a quantitative correlation between road safety and traffic congestion level.

In fact, the HCM[9] can predict the expected LOS, if the traffic demand and available road capacity are known. However, the sole HCM does not consider the interaction that could exist between the crashes and their effect on traffic.

Using the Highway Capacity Manual (2000)[9], Kononov et al. (2008)[8] were able to estimate the extent of Levels of Service ranges in the SPF graphs, according to traffic variations during rush hours. Comparing the width of LOS area, varying the traffic flow at the peak hour with regard to SPFs calculated for both total crashes and only fatal and injury crashes, the authors came to the following observations:

- the total crashes and fatal/injury crashes increase with the AADT;
- it is safer to travel on urban freeways which are characterized by a LOS-C or better LOS during the peak hour, rather than on roads with higher levels of congestion.

The figure 10.5 shows the operational flow conditions on a road segment when the flow rate of a basic freeway segment changes. In addition, five levels of service can be defined (Level of Service LOS), from "A" to "E" (LOS A represents the best conditions, while LOS E the worst). From the figure it is also highlighted that, on freeways, drivers slightly vary their speeds passing from LOS A to LOS D (in the following figure, significant changes in speeds across LOS C and LOS E are highlighted with red lines).

As noted from the previous figure, there are more vehicles in a given space while they travel at similar speeds. However, the authors suggest that perception and reaction times stay constant as well as the vehicle characteristics[6].

Considering these elements, it can be again concluded that more crashes can happen, and then crash rates can change. A quantitative relation between road safety and the traffic congestion level has then been introduced (see the following figure).

[9] AASHTO (2000), *Highway Capacity Manual (HCM)*, Transportation Research Board, National Research Council, Washington D. C., USA.

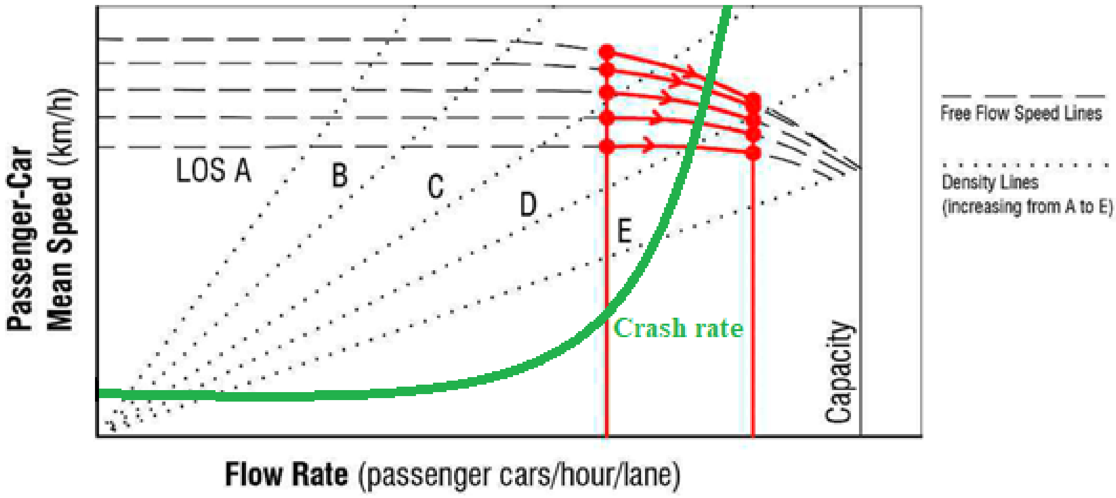

Fig. 10.5: Overlap between the speed-flow-density/LOS curve and the crash rate curve (based on Kononov et al. 2012[7] *and HCM (2000)*[9] *for the LOS concept).*

In order to improve the capacity of a freeway it is necessary to increase the number of lanes. Kononov et al. (2012)[7], in contrast to common engineering beliefs, report that most research shows that the crash rate increases as the number of lanes increase. They explain this increase in crash rate by the increase in vehicles conflicts due to lane changes.

According to the HCM (2010)[10], the number of lanes on a freeway segment can increase operating speeds and then the probability that a driver will overtake slower vehicles.

A greater chance of overtaking vehicles may increase the average traffic flow speed, as well as the speed gap between vehicles and a higher number of crashes due to lane changes[8]. In brief, the number of possible crashes in one travel direction can be expressed as a function of the freeway number of lanes.

At the same time, increasing the number of lanes, the traffic volume also increases, which may lead to even more crashes. For instance, in the example of Figure 10.6 the crash frequency may triple if the traffic volume doubles.

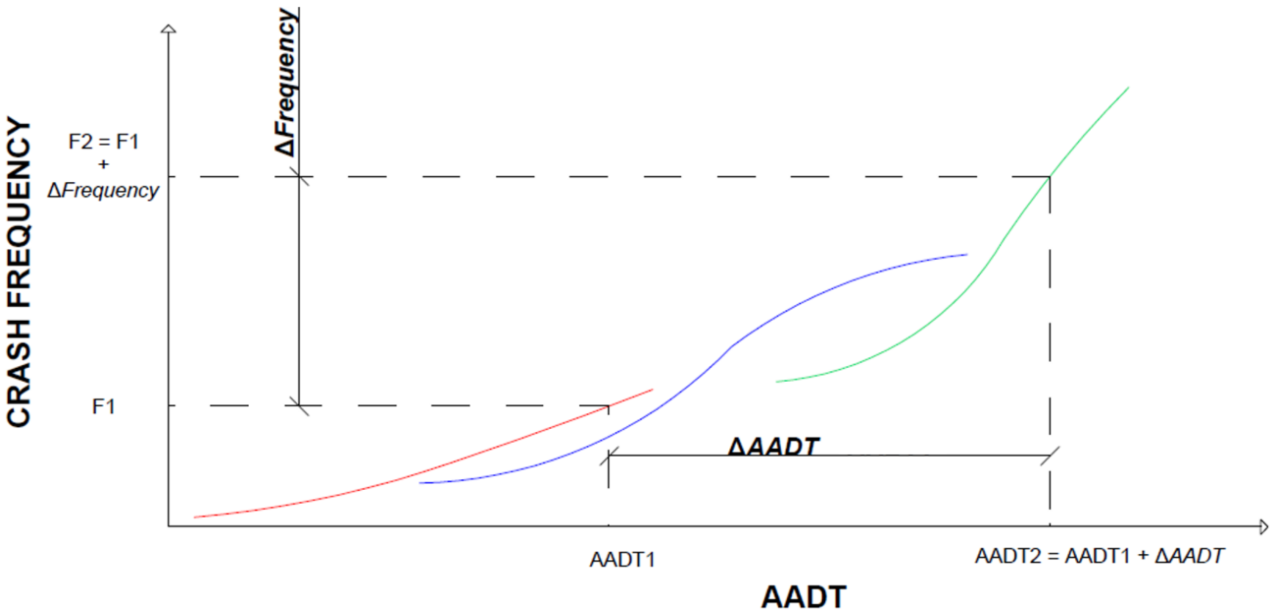

Fig. 10.6: Relationship between crash frequency and number of lanes, from 4-lane freeways (in red) to 8-lane freeways (in green) (based on Kononov et al., 2008[8]*).*

[10]AASHTO (2010), *Highway Capacity Manual* (HCM), Transportation Research Board, National Research Council, Washington D. C., USA.

 ISBN 9788835111863

The relationship between the number of lanes (n) and conflicts (Cn) is expressed by the following equations[8]:

- for n = 2

$$C_n = n(n-1) \qquad \text{(Eq. 10-2)}$$

- for n > 2

$$C_n = n(n-1) + \frac{n!}{3!(n-3)!} \qquad \text{(Eq. 10-3)}$$

Where:

- C_n = possible conflicts which could happen while changing lane in one direction;
- n = number of lanes for each travel direction.

For example, in a 8-lane carriageway without medians, 4 for each travel directions, there would be 16 potential conflicts according to Eq. 10-3. If the same road has a median barrier to divide the 4 lanes into 2+2 lanes, the expected number of total conflicts would become 4 according to Eq. 10-2, Eq. 10-3, showing a noticeable decrease.

Therefore, taking into account the safety performance, it can be observed that[8]:

- the increse in the number of general lanes brings safety issues as side effect;
- traffic density is temporarily reduced when road capacity is increased due to more lanes, which creates a temporary improvement in driving conditions and safety.

However, with reference to the above listed last point, it can be noted from Fig. 10.6 that a freeway SPF can show a sigmoid shape, with a smooth increase for low volumes, a steep increase for higher volumes (because of the higher conflicts) and a smooth increase again for high volumes in case of congested traffic. Hence, for a given traffic volume (e.g., the AADT1 in Fig. 10.6) the temporary safety improvement (reduction in crash frequency with reference to the blue curve) provided by the increased lanes can be recovered if the traffic volume increases again[8].

10.3 LOSS and crash diagnosis

Once a safety problem is identified through the LOSS method, the understanding of the specific safety issues related to the road site should be diagnosed. To this aim, a general framework has been reported by Kononov et al. (2003)[4] to conduct the diagnostic analysis of road safety problems for different road types in various scenarios. In particular, 84 parameters were defined to provide a baseline framework for the diagnostic analysis of different road types in both urban and rural areas. Starting from the fact that road crashes can be considered as random Bernoulli events, the authors elaborate that it is possible to identify deviations from the random statistical process through the calculation of the cumulative probability observed for each of these parameters.

The 84 parameters belong to 11 general categories:

- crash type;
- crash severity;
- crash location;
- road conditions;
- travel direction;
- lighting conditions;
- type of vehicle;
- human factors;
- driver conditions;
- weather conditions;
- time of day.

The 84 standard parameters were also grouped by three ranges of AADT values: low, medium, and high. The subdivision of diagnostic parameters according to the AADT may improve the capability to identify more accurately some specific crash patterns.

In fact, the authors report that the analysis of the SPF should be used in combination with an appropriate diagnostic investigation, such as the so-called “pattern recognition” algorithm, useful for estimating the size of road safety problems at specific points.

The following figure combines the different LOSS areas with the AADT ranges, which are suggested to be used for diagnostic investigation.

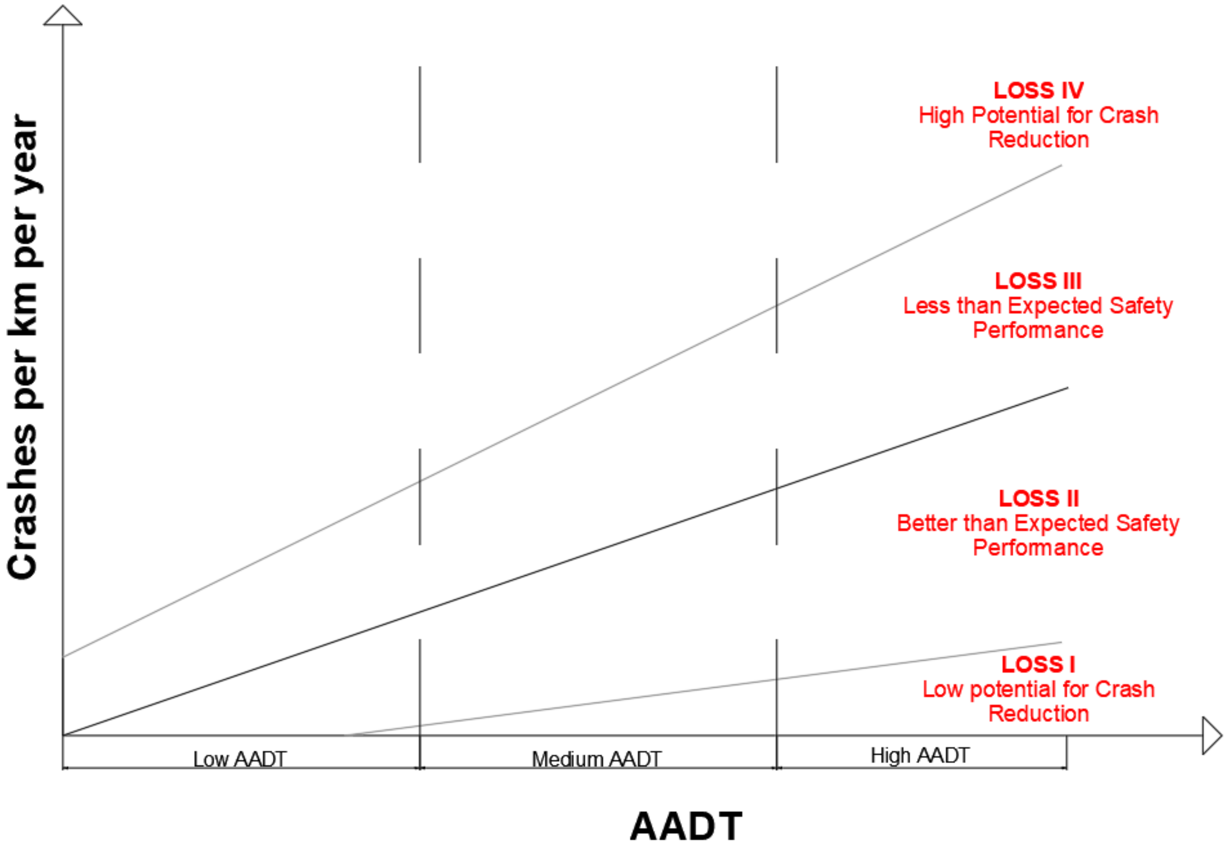

Fig. 10.7: Combination of LOSS areas and AADT ranges for diagnostic purposes (Based on Kononov et al., 2003[4]).

In the study by Kononov et al. (2003)[4] a diagnostic case was examined after a LOSS analysis on a rural mountainous road section having more crashes than expected (in particular fixed-object crashes, even if not particularly related to bad weather/pavement conditions). The in-depth analysis of the crash phenomena, the road geometry and the surrounding environment have revealed important crash-related aspects which were used to suggest appropriate countermeasures (e.g., safety barriers, automatic speed control and road signs).

10.4 References

AASHTO (2000), *Highway Capacity Manual (HCM)*, Transportation Research Board, National Research Council, Washington D.C., USA.

AASHTO (2010), *Highway Capacity Manual (HCM)*, Transportation Research Board, National Research Council, Washington D.C., USA.

AASHTO (2010), *Highway Safety Manual, First Edition*, Transportation Research Board, National Research Council, Washington D.C., USA.

Hauer E., Persaud. B. (1997), *Safety Analysis of Roadway Geometric and Ancillary Features*, Transportation association of Canada.

Kononov J., Allery B. (2003), "Level of Service of Safety: Conceptual Blueprint and Analytical Framework", *Transportation Research Record: Journal of the Transportation Research Board*, 1840(1), 57-66

Kononov J., Bailey B., Allery B. (2008), "Exploratory Examination of the Functional Form of Safety Performance Functions of Urban Freeways", *Presentation at the 2008 TRB Annual Meeting*.

Kononov J., Bailey B., Allery B. (2008), "Relationships Between Safety and Both Congestion and Number of Lanes on Urban Freeways", *Transportation Research Record: Journal of the Transportation Research Board*, 2083(1), 26-39.

Kononov J., Lyon C., Allery B. (2011), "Relation of Flow, Speed, and Density of Urban Freeways to Functional Form of a Safety Performance Function", *Transportation Research Record: Journal of the Transportation Research Board*, 2236(1), 11-19.

Kononov J., Reeves D., Durso C., Allery B. K. (2012), “Relationship between freeway flow parameters and safety and its implication for adding lanes”, *Transportation research record*, 2279(1), 118-123.
Kononov, J., Durso, C., Lyon, C., & Allery, B. (2015), “Level of service of safety revisited”, *Transportation research record*, *2514*(1), 10-20.

11. Proposal of the new design protocol

Since the Italian guidelines have not been fully implemented yet (as in some other countries), it is possible to draw up a new hypothesis about an operational framework. This hypothesis concerns adapting the HSM to the local case studies, always looking at the upcoming implementation of the Italian guidelines. However, this protocol, here further applied to Italian case studies in next chapters, could be implemented regardless of the specific country.

The list below provides the steps for a protocol of design of safety interventions on existing roads:

- identifying the road network;
- identifying the homogenous road segments;
- retrieving crash data and traffic data;
- identifying the SPFs, CMFs and Cc;
- identifying the available metrics;
- drawing up all the ranking lists and the final ranking list too, with the different methods using also the SPFs, CMFs and Cc;
- planning and making: inspections (as based on local guidelines), surveys, reports and drawings of the actual conditions, crash diagrams, condition diagrams, comparisons with the standards, friction diagrams, reports about drivers' population (considering also familiar and unfamiliar users);
- identifying factors involved in crash occurrence and diagnostic;
- drawing preliminary project documents about all the possible countermeasures and designing alternatives;
- comparing the alternatives basing on the optimal choice according to the SPFs, CMFs and Cc;
- drawing up the executive project documents.

This procedure can be applied for both the urban and the rural context, as it will be shown in detail in Chapter 13. One of the most relevant differences between the urban and rural context is the scale of the problem: speeds at which vehicles travel and the interactions with other road users, such as bicyclists and pedestrians, are largely different. Their presence should be not neglected thanks to appropriate SPFs and CMFs developed for urban cases.

11.1 References

AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

Ministerial Decree n. 137 of 2 May 2012: *Guidelines for the management of road infrastructure safety pursuant to art. 8 of Legislative Decree no. 35 of 15 March 2011 - Linee guida per la gestione della sicurezza delle infrastrutture stradali ai sensi dell'art. 8 del decreto legislativo 15 marzo 2011, n. 35*.

12. *Application of the HSM method in Europe*

In this chapter, some practical aspects related to the application of the predictive method proposed by the Highway Safety Manual in Europe are shown. In particular, possible issues in applying SPFs are discussed in detail. Most of these issues are valid regardless of the specific country, i.e., they are not specific to Italy or any other country.

The first important issue while applying the HSM predictive method is the choice between calibrating an existing SPF (e.g., HSM SPFs) for the local context or developing a local SPF. This topic is discussed in detail in the next section.

12.1 Calibration of existing SPFs (transferred functions) or development of local SPFs

The following question is repeatedly posed in the HSM and generally in the international debate: is it more appropriate to conduct local calibration studies using some known SPFs or to develop native local SPFs based on database of traffic, crashes and road segment characteristics?

The answer to this question is not immediate. In fact, there are different opinions in the literature (as further described later in this chapter) on the possibility of transferring, for example, the basic SPFs proposed in the HSM (developed in the United States) to very different contexts, such as Europe.

Indeed, differences in road context, in drivers' populations and behaviours, and in the availability of crash database may result in unreliable transferred SPF from a context to another. In order to reduce the amount of calculation and analyses deriving from these discrepancies, it is possible to calibrate the transferred SPF (Transferred Function -TF-) by adapting the outcomes to local conditions.

The basic calibration method is provided by the HSM. The proposed local calibration factors are expressed as a ratio between the total observed crashes and the total predicted crashes for a chosen road type. However, note that for large samples of sites, some different and more reliable techniques are available[1,2,3].

The calibration of transferred SPFs may be less demanding than the local estimation (in general a limited range of variables and possible combinations of them is considered, due to the small sample size), but it could be less reliable because it implies accepting the same variables included in the baseline model for the specific local context. This possible lower precision may be still shown even after having improved crash estimates by using CMFs (crash Modification Factors).

Instead, if there is a great availability of data and the analysis requires a great amount of information as well as remarkably precise models, the development of a local SPF (Local Function -LF-) can be the optimal solution, because it is related to local road sites and its variables depend on the specific case and conditions. However, the choice of local SPFs does not necessarily imply such remarkable improvements in crash estimations, to be justified. Moreover, a local SPF often requires hard computational tasks. This because a local SPF (LF) considers different crash-related variables, within a single model to predict crash frequencies. In fact, LFs rely on specific variables important for the study, giving more precise details about the crash

[1] Bahar G., & Hauer E. (2014), *User's guide to develop highway safety manual safety performance function calibration factors*, National Cooperative Highway Research Program.

[2] Xie F., Gladhill K., Dixon K., Monsere C. (2011), "Calibration of Highway Safety Manual Predictive Models for Oregon State Highways", *Transportation Research Record: Journal of the Transportation Research Board,* 2241, 19-28.

[3] Shin H. S., Dadvar S., Lee Y. J. (2015), "Results and Lessons from the Local Calibration Process of the Highway Safety Manual for the State of Maryland", *Transportation Research Board 94th Annual Meeting* (No. 15-4643).

phenomenon. However, if variables lack of statistical significance, they could affect badly the results highlighting insignificant tendencies. In summary, LFs are usually encouraged (AASHTO, 2010[4]) but they require greater efforts, not always justified by their performances (a benefit-cost estimation may help to prefer LFs against TFs).

Hence, sometimes using TFs might be the right choice, especially in case of abundant reference SPFs, good quality of data and modelling techniques used, and of similarity between the investigated areas and the one in which the reference SPF has been developed.

Another benefit of using TFs is that TFs are easy to be handled especially by not-expert users, who should follow simple guidelines to apply them. Nevertheless, the choice between TF and LF is not standardized due to the fact that it is impossible to know a priori if the LF will outperform the TF. Moreover, this issue (LF vs TF) is not strongly documented for different contexts: several reference SPFs exist in North America, while not in Europe.

An example of assessment of TFs versus LFs as based on two European case studies (Italy and Scotland) was performed by Intini et al. (2019)[5]. After having estimated crashes with both the possible approaches for the specific cases considered, the authors have found that differences in the outputs between LFs and TFs were not significant. This confirms that LFs could be only justified for a large sample size (avoiding biased estimation from a small sample size), otherwise a TF can be acceptable in terms of time, cost efforts and reliability of results. Although calibrating TFs is easier, statistically significant factors considered for calibration (e.g., traffic ranges, regions) may be false positive when checked against results from LFs.

However, this concern cannot be avoided by using LFs in case of small sample sizes: LFs could not provide significant improvements. So, when the sample size is small and very detailed datasets of crashes, traffic volumes and other variables are not available, a TF may be the better choice. In addition, the calibration of SPFs (TF) is absolutely needed when basing crash predictions on the HSM SPFs. In fact, in that case, the straight use of basic SPFs present in the HSM manual could lead to relevant errors in the prediction.

This concept can be easily understood from the following figure. The figure shows in orange the SPF calibrated for the Italian context as obtained from the study by Colonna et al. (2016)[6] for rural road sections (two-lane undivided rural roads, with one lane per direction). The basic SPF present in the HSM manual for the same type of roads is reported in blue. The traffic range for which both functions are valid is that defined in the HSM, i.e., an AADT between 0 and 17800 vehicles/day.

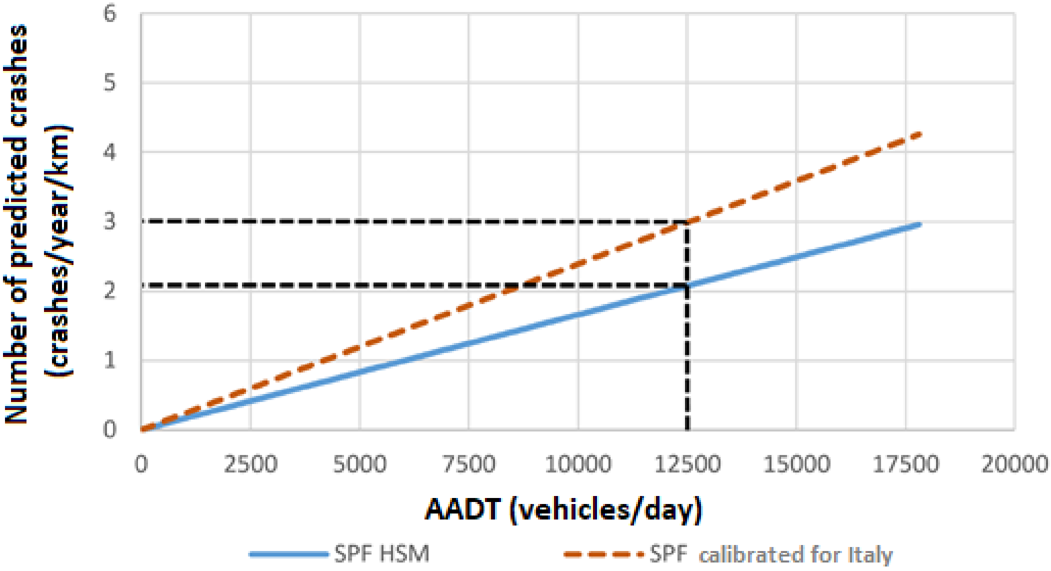

Fig. 12.1: Comparison between HSM base and calibrated SPFs for Italian two-lane rural roads (based on Colonna et al., 2016[6]).

[4]AASHTO (2010), *Highway Safety Manual, First Edition,* Transportation Research Board, National Research Council, Washington D.C., USA.

[5]Intini P., Berloco N., Binetti R., Fonzone A., Ranieri V., & Colonna P. (2019), "Transferred versus local Safety Performance Functions: A geographical analysis considering two European case studies", *Safety Science*, 120, 906-921.

[6] Colonna P., Berloco N., Intini P., Perruccio A., Ranieri V., Vitucci V. (2016), "Variability of the Calibration Factors of the HSM SPF with Traffic, Region and Terrain. The Case of the Italian Rural Two-Lane Undivided Road Network", *Compendium of Papers of the 95th Annual Meeting of the Transportation Research Board,* Washington, D.C., USA.

In this case, the prediction made using the basic HSM function (in blue) underestimates the crash frequency compared to the calibrated function (in orange), obtained with the help of 398 Italian sites. In the example shown in Figure 12.1, with an AADT value equal to 12,500 vehicles/day, the average predicted crash frequency is about 2 crashes/km/year according to the basic SPF and about 3 crashes/km/year according to the calibrated SPF.

This helps to understand how much the use of a basic SPF instead of a calibrated SPF may lead to significant errors in crash prediction.

In the following figure, on the other hand, the development of a simplified native SPF (considering only the traffic variable) based on the data available for the calibration (the same study cited above) is shown as an example.

Considering a traffic range with a higher upper boundary, a significantly different functional form than the simple linear function provided by the HSM for the two-lane rural road segments could be chosen. In this case, this function better fits the available data.

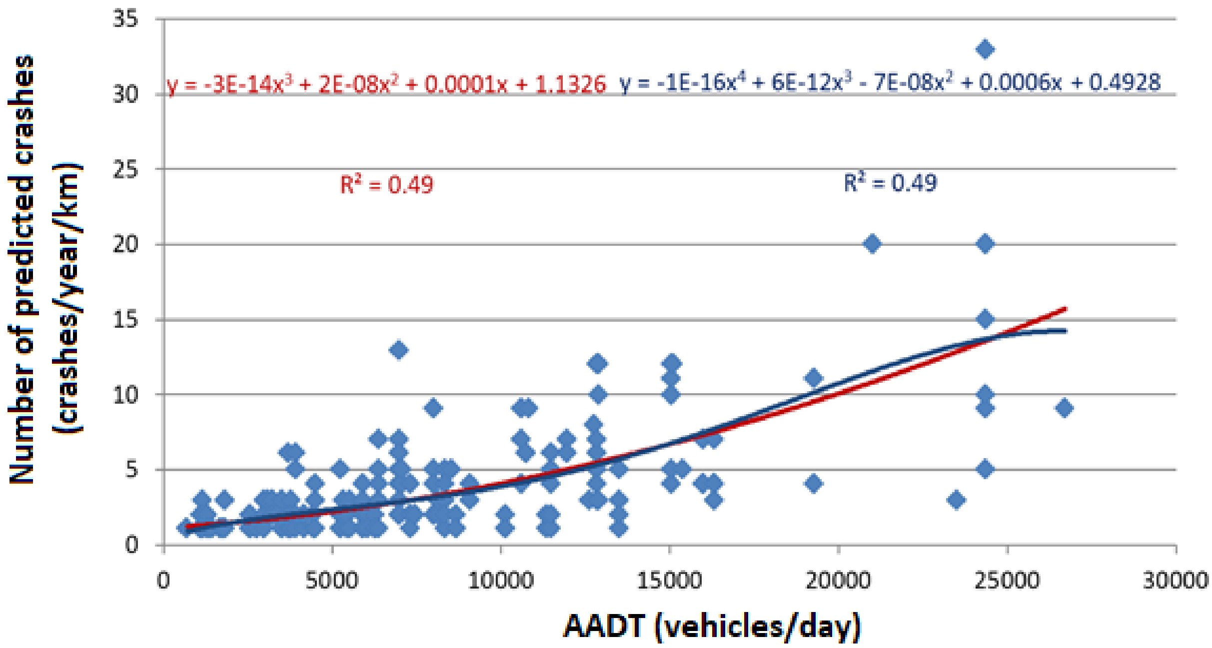

Fig. 12.2: Example of native SPF development with different functional forms.

However, the description of techniques related to the development of native Performance Functions is beyond the scope of this chapter. For further details, please refer to the guide for the development of SPFs prepared by the University of North Carolina for the U.S. FHWA[7] or to the Hauer's monograph (Hauer, 2015[8]). An organic portrait of the above explained concepts to the Italian case is provided below. The two following paragraphs show the estimation of calibration coefficients and local SPFs.

12.2 General remarks about the calibration of SPFs

The predicted crash frequency is based on the following equation, according to HSM method:

$$N_{predicted} = N_{SPF}\ x\ CMF_1\ x\ CMF_2 \ldots x\ CMF_n\ x\ C_c \qquad \text{(Eq. 12-1)}$$

[7] Srinivasan R., Bauer K. (2013), *University of North Carolina, Highway Safety Research Center. Safety Performance Function Development Guide: Developing Jurisdiction-Specific SPFs*, Final Report. Prepared for Federal Highway Administration Office of Safety.

[8] Hauer E. (2015), *The Art of Regression Modeling in Road Safety*, Springer, New York, USA.

N_{SPF} is the number of crashes predicted by the Safety Performance Function under baseline conditions, CMFs are crash modification factors and C_c is the calibration coefficient that considers the application of the SPF model in different jurisdictions or in a different time period.

The calibration coefficient is the ratio between the sum of observed crash frequencies and the sum of predicted average crash frequencies for the same sites, during the same period of time and using the appropriate predictive models (e.g., those provided by the HSM).

$$C_c = \frac{\sum_{all\ segments} Observed\ number\ of\ accidents}{\sum_{all\ segments} Predicted\ number\ of\ accidents\ (uncalibrated\ SPF)} \qquad \text{(Eq.12-2)}$$

If the coefficient is greater than 1, then the analysed roads will have more crashes than the roads that were used to develop the SPF and vice versa.

Crash frequencies on road segments or similar intersections can considerably vary between States, but also between geographical regions belonging to the same State or different sub-networks in the same region. For this reason, the Safety Performance Function calibration for local conditions is necessary. Indeed, different regions may differ in climatic conditions, driver populations, crash recording methods and thresholds, other characteristics. All these differences could be considered as road safety issues because they concern the interaction between different elements related to human, road, environmental, traffic and vehicle factors.

The HSM provides the procedure for developing calibration coefficients. Some studies in literature document the results of the calibration process in the United States (applied in different States, e.g. Oregon and Maryland[9,10]) and discuss the transferability of HSM SPFs to different countries (e.g. Canada[11] and Italy[12,13]). These studies have shown a wide variability in the calibration coefficient, also due to the different road network types on which the calibration process has been applied. A recent publication by AASHTO (NCHRP 20-07-332)[4] also highlighted the importance of considering the variability of the calibration coefficient in relation to other characteristics (crash severity, traffic ranges, segment lengths, geographical regions, terrain type). It proposed some techniques to evaluate the calibration results, in addition to what is described in the HSM. Further guidelines were then provided in the more recent report by Lord et al. (2016)[14].

In the following paragraphs, some practical and scientific aspects related to the implementation of the HSM and subsequent documents on the calibration of SPFs to local conditions are discussed.

12.2.1 Calibration of SPFs: Italian studies

In absence of local SPF, applying the HSM method is possible due to the calibration coefficients for each local context and for each road type under investigation. According to the case studies shown in this book, some examples of SPFs calibration are here shown for Italy. Note however that the calibration process shown in the HSM can be eventually applied everywhere, as based on the same hypotheses. The following table briefly highlights the results of the calibration studies run in Italy for different purposes and for different road types considering the HSM publication (2010)[4]. The investigated roads were either rural two-way two-lane roads or rural divided freeways.

[9] Dixon K., Monsere C., Xie F., & Gladhill K. (2012), *Calibrating the future highway safety manual predictive methods for Oregon state highways* (No. FHWA-OR-RD-12-07), Oregon. Dept. of Transportation. Research Section.

[10] Dadvar S., Lee Y. J., & Shin H. S. (2020), “Improving crash predictability of the Highway Safety Manual through optimizing local calibration process”, *Accident Analysis & Prevention*, 136, 105393.

[11] Persaud B., Saleem, T., Faisal S., Lyon C., Chen Y., Sabbaghi A. (2012), “Adoption of Highway Safety Manual Predictive Methodologies for Canadian Highways”, *Proceedings from the 2012 Conference of the Transportation Association of Canada*, Fredericton, New Brunswick.

[12] Sacchi E., Persaud B.; Bassani M. (2012), “Assessing international transferability of Highway Safety Manual crash prediction algorithm and its components”, *Transportation Research Record: Journal of the Transportation Research Board* 2279, 90-98.

[13] La Torre F., Domenichini L., Corsi F., Fanfani F. (2014), “Transferability of the Highway Safety Manual Freeway Model to the Italian Motorway Network”, *Transportation Research Record: Journal of the Transportation Research Board*, (2435), 61-71.

[14] Lord D., Geedipally S.R., Shirazi M. (2016), *Improved guidelines for estimating the Highway Safety Manual calibration factors* (No. ATLAS-2015-10), University Transportation Centers Program (US).

Tab. 12.1: Summary of the calibration studies for the Italian context and the obtained calibration coefficients, Cc.

Road type	*Geographical area*	*Authors*	*Method*	*Dataset*	*Years*	*Cc*
Freeways	Sicily	Cafiso et. al. (2012)[15]	HSM	47 segments (58 km), 314 fatal/injury crashes	2005 -2008	1.26
Freeways	Italy	La Torre et al. (2014)[13]	HSM + NCHRP (17-45)	56 segments (700 km)	2005 -2009	1.52 (MV, FI)* 1.19 (MV, PDO)* 0.36 (SV, FI)* 0.64 (SV, PDO)*
Two-lane rural roads	Tuscany (Arezzo)	Martinelli et al. (2009)[16]	HSM (Draft Chapter)	938 km 402 crashes	2002 -2004	0.37
Two-lane rural roads	Piemonte (Turin)	Sacchi et al. (2012)[12]	HSM	242 segments (115 km), 236 fatal/injury crashes	2005 -2008	0.44
Two-lane rural roads	Italy	Colonna et al. (2016)[6]	HSM + NCHRP (20-07)	398 segments (220 km), 422 fatal/injury crashes	2008 -2012	1.44

*MV = multi-vehicle, SV = single-vehicle, FI = fatal+injury crashes, PDO = property damage only crashes.

12.2.2 Influences of traffic variables, region and terrain elevation on the calibration coefficient

The calibration of SPFs is necessary to assess the variability of weather conditions, drivers' behaviours, drivers' population, thresholds of crash reporting/recording methods and other characteristics, in order to obtain reliable results of crash frequency on the investigated road segments and intersections. The main point is the following: how large a geographic area can be, so that a unique calibration coefficient for the crash prediction can be used, according to the HSM method?

The differences between countries and, within the same country, between different regions might influence the quantitative road safety assessment. There is a wide set of studies run in the United States for the calibration in different States. However, it is worthy to ask whether a unique calibration coefficient for the state of Maryland (area: 30'000 km^2, population: 6 million of inhabitants) has the same validity of the unique coefficient for the state of Texas (area: 700'000 km^2, population: 28 million of inhabitants). And for Italy, which has the half of the extension of Texas, but with an almost doubled population and so a great density of population, is a single calibration coefficient acceptable? This should involve assuming unique weather conditions, unique drivers' behaviours, unique recording methods for crashes and other characteristics for the whole country. However, this assumption neglects all the variability owned by a country like Italy, as for instance the terrain elevation (e.g. Alps and Apennines areas compared to completely flat areas).

Either the HSM or the NCHRP 20-07 (332)[4], which was the early guideline for calibration, focus on the above explained topic, pointing out the need for considering the influence of regionality and geomorphic characteristics (as well as traffic ranges, as already previously explained) on the calibration coefficient.

However, the two references for the calibration procedures do not mention a unique procedure or precise methods on how to consider the aforementioned variables and on how to divide the sites accordingly. Further detail on calibration procedures were after provided by Lord et al. (2016)[14].

The results from two studies (Colonna et al., 2016[6]; Intini et al., 2019[5]) are here shown to assess the influence of traffic, region and terrain on the calibration coefficient for Italian two-lane rural roads.

These studies were based on a dataset of fatal/injury crashes, corresponding to the KAB group – excluding the possible injuries, C – of the KABCO scale (HSM). Nevertheless, the SPF proposed by the HSM for this road category is valid for total crashes, so it is possible to adopt the calculated calibration coefficient in these studies, for the total crashes. In fact, it is possible to obtain the total crashes starting from the fatal/injury crashes, using the available percentages of crashes according to severity and hence, going from the predicted crash frequency for KAB crashes to the predicted crash frequency for KABCO crashes. If reliable local data are not available, the percentages provided by the HSM can be used. It is blatant the reduction of approximation of this procedure using the coefficients of calibration derived from the prediction of solely fatal/injury crashes.

[15] Cafiso S. Di Silvestro G.; Di Guardo G. (2012), "Application of Highway Safety Manual to Italian divided multilane highways", *Procedia-Social and Behavioral Sciences*, 53, 910-919.

[16] Martinelli F., La Torre F., Vadi P. (2009), "Calibration of the Highway Safety Manual's Accident Prediction Model for Italian Secondary Road Network", *Transportation Research Record* 2103.

Colonna et al. (2016)[6] calculated the calibration coefficients for several sub-sets of traffic volumes (less and greater than 10'000 vehicles/day), terrain elevation (rolling and flat terrain) and of regions (according to a macro-division: Northern Italy and Centre/South Italy), as shown in next table.

The reliability of the obtained calibration coefficient was assessed by using the coefficient of variation:

$$c_V[C_c] = \sigma[C_c]/C_c \qquad \text{(Eq. 12-3)}$$

where:
$c_V[C_c]$ = coefficient of variation of the variable C_c, that is the calibration coefficient,
$\sigma[C_c]$ = standard deviation of the variable C_c,

Tab. 12.2: Calibration coefficients for different combinations of traffic, region and terrain type (based on Colonna et al., 2016[6]).

Variable	*Sub-group*	*AADT range*	*Number of sites*	*Cc**	*Cv [Cc]*
-	Global (country-level)	Overall	398	*1.44*	0.07
		< 10000	316	*1.19*	0.09
		10000-17800	82	*1.75*	0.10
Terrain Type	Rolling terrain	Overall	237	*1.38*	0.11
		< 10000	200	*1.22*	0.13
		10000-17800	37	1.62	0.19
	Flat terrain	Overall	161	*1.49*	0.08
		< 10000	116	*1.17*	0.11
		10000-17800	45	*1.82*	0.11
Region (macro)	Northern Italy	Overall	112	*1.66*	0.10
		< 10000	51	1.39	0.21
		10000-17800	61	*1.71*	0.11
	Centre-South Italy	Overall	286	*1.29*	0.08
		< 10000	265	*1.16*	0.09
		10000-17800	21	1.81	0.20
*Macro Region/ Terrain Type^*	Northern Italy/ Rolling terrain	Overall	50	1.73	0.19
		< 10000	23	1.36	0.33
		10000-17800	27	1.84	0.22
	Northern Italy/ Flat terrain	Overall	62	*1.62*	0.11
		< 10000	28	1.41	0.26
		10000-17800	34	*1.67*	0.13
	Centre-South Italy/ Rolling terrain	Overall	187	*1.18*	0.13
		< 10000	177	*1.19*	0.14
		10000-17800	10	1.10	0.37
	Centre-South Italy/ Flat terrain	Overall	99	*1.37*	0.10
		< 10000	88	*1.13*	0.12
		10000-17800	11	2.28	0.20
Administrative Region	Basilicata	Overall	71	0.46	0.35
		< 10000	68	0.43	0.35
		10000-17800	3	0.65	1.40
	Calabria	Overall	43	1.61	0.16
		< 10000	36	0.99	0.17
		10000-17800	7	2.73	0.34
	Campania	Overall	28	1.03	0.26
	Emilia Romagna	Overall	15	1.24	0.23
		< 10000	3	2.07	1.03
		10000-17800	12	1.20	0.23
	Lombardia	Overall	33	1.46	0.29
		< 10000	16	0.87	0.36
		10000-17800	17	1.71	0.45
	Molise	<10000	9	0.90	0.53
	Puglia	Overall	112	*1.26*	0.12
		< 10000	101	*1.24*	0.12
		10000-17800	11	1.34	0.30
	Umbria	< 10000	23	2.60	0.25
	Veneto	Overall	64	*1.90*	0.12
		< 10000	32	*1.71*	0.14
		10000-17800	32	1.94	0.20

*Bold coefficients are referred to a sub-set composed of at least 30 sites, with a coefficient of variation less than 0.15.
^Rolling terrain: sites with terrain elevation greater than 400 m belong to this category; Flat terrain: sites with elevation less than 400 m. Northern Italy: includes sites in Lombardy, Emilia Romagna and Veneto; Centre-South Italy: includes sites in Basilicata, Puglia, Campania, Calabria, Molise and Umbria.

The calibration coefficients listed in table 12.2 in bold could be used for practical applications at sites which show one of the combinations in table. In fact, these coefficients have been calculated for at least 30 sites, as stated by the HSM, and they show a coefficient of variation (a metric of the reliability of Cc), at least equal to 0.15, as suggested by the NHCRP 20-07. In any case, further remarks are stated in the cited reference.

Similar research questions were addressed in the study by Intini et al. (2019)[5]. In this research, datasets of traffic volumes and crashes (in the period 2008-2012) on two-lane rural roads were used. The secondary intersections were not considered, because of the traffic volume assumed as constant on the segment. The total length of the 74 segments included in the study is 213 km. Italy was divided in two regions in order to synthetically capture the influence of socio-economic and possible driving behaviour differences: Northern Italy and South/Centre Italy. The terrain type was also considered: the threshold between flat and rolling terrain was set again equal to 400 m above mean sea level (as in Colonna et al., 2016[6]). The traffic threshold between different traffic ranges was set to 10'000 vehicles/day because the HSM SPF increasing tendency may be more than linear for traffic volumes approximately greater than 10'000 vehicles/day[12]. The road segments included in the study have a significant length and then they were divided in sub-sections having internal homogenous geometric characteristics. Hence, all the assessments stated by the HSM in order to provide a reliable calibration of the SPF (30-50 homogenous road segment, 100 crashes/year over the total sample of sites, at least 3 recent years of crash data) were generally respected (except for a slightly lower number of crashes).

The reliability of the calibration coefficient was again set according to the coefficient of variation: if cv {Cc} < 0.20, the Cc can be considered as reliable. If the minimum number of road sites required for calibration is not achieved, the use of a local SPF is advised, as well as when the calibration coefficient is not reliable.

Tab. 12.3: Calibration coefficients for different region and traffic conditions (Intini et al., 2019[5]).

Variable: Region	*AADT Ranges*	*Cc*
Overall	Overall	1.44
	< 10,000	1.19
	≥ 10,000	1.75
Northern Italy	Overall	1.66
	< 10,000	*1.39*
	≥ 10,000	1.73
Centre-South Italy	Overall	1.29
	< 10,000	1.16
	≥ 10,000	*1.81*

*Cc coefficients in italics are deemed less reliable due to either related number of segments < 30 or cv [Cc] ≥ 0.20.

The results seem reliable except for the low traffic coefficient in Northern Italy and for high traffic volumes in the other region. A regional effect can be noted in the outputs of the HSM calibration. The overall factor for Northern Italy is considerably higher than for Centre-South Italy, and indeed a regional calibration factor was deemed necessary considering the method proposed by Lord et al. (2016)[14]. The difference between the regional coefficients may be attributed to the high percentage of high traffic sites for Northern Italy, which may have led to the notably high Cc for Northern Italy. Whereas, when comparing high traffic ranges, no consistent differences are noted.

After having shown these examples of application, it is important to remark that, nowadays, the scientific debate is still open about the number, the importance/type of variables to be considered and how they must be used in the calibration process. Concerning the country, macro-regional and regional levels, the use of precise and different calibration coefficients could have a deep impact on priority choices (as the priority choice between an intervention in Lombardia or in Campania could be, in case of Italian regions). However, it does not have influence on the choice of single projects already identified as necessary. In fact, in this case the calibration coefficient used to calculate the benefits of the crash reduction is similar for each set of countermeasures.

Instead, significantly different calibration coefficients for different regions may affect priority choices between projects in different regions.

12.2.3 Example of application of calibration coefficients

Assuming that after the road network screening in the jurisdiction, the relevant agency decides to prioritize road safety interventions on a two-lane rural road, whose characteristics are the following:

- L = 3 Km,
- AADT =9,500 vehicles/day,
- Crash number = 10 crashes over 5 years (KAB severity according to the KABCO scale by HSM),
- Region = Puglia.

The further assumption is that the road geometry is equal to the baseline conditions stated by HSM. This implies that the product of CMFs is equal to 1 in the current configuration, and the road segment is homogenous.

After the diagnosis and the selection of countermeasures stages, two proposals have been stated for the whole road segment, characterized by the following CMFs for the countermeasures in each of the two solutions:

- Intervention 1: Total CMFs product = 0.82, lifetime = 10 years;
- Intervention 2: Total CMFs product = 0.80, lifetime = 10 years.

The calculation of the mean expected crash frequency is the next step, for both the current configuration and the post-intervention scenario, under the assumptions of the considered example. The expected crash frequency is preferable to the calibrated predicted crash number from the HSM SPFs, as they consider the observed crashes too and it overcomes the RTM bias (see chapter 7).

$$N_{Expected,current} = w \times N_{Predicted,current} + (1-w) \times N_{Observed} \qquad \text{(Eq. 12-4)}$$

$$N_{predicted,KAB} = C_c \; x \; N_{spf,total} \; xCMF_1 xCMF_2 \dots xCMF_n \; x \; (\% \; KAB) \qquad \text{(Eq. 12-5)}$$

In the specific case:

- C_x = 1.24, from Table 12.2, Puglia, and for AADT range with less than 10’000 vehicles/day;
- $N_{SPF,\,total}$ = number of crashes obtained by the SPF for two-lane rural roads (based on the HSM SPF) = 4.73 crashes/year;
- CMF_1 x CMF_2 ... x CMF_n = 1, as stated for the assumed scenario;
- % KAB = 0.176, default KAB percentage over the total number of crashes provided by the HSM.

Hence:

$$N_{predicted,KAB} = 1.24 \; x \; 4.73 \; x \; 1 \; x \; 0.176 = 1.03 \, \frac{KAB \; crashes}{year} \qquad \text{(Eq. 12-6)}$$

$$N_{predicted,KAB,period \; of \; investigation} = 1.032 \; x \; 5 = 5.16 \;\; KAB \; crashes \qquad \text{(Eq. 12-7)}$$

$$N_{Observed,period \; of \; investigation} = 10 \; KAB \; crashes \qquad \text{(Eq. 12-8)}$$

$$w = \frac{1}{1+K \; x \; \sum N_{Predicted}} = \frac{1}{1+\left(\frac{0.38}{3 \; km}\right) x \; 5.16 \; KAB \; crashes} = 0.60 \qquad \text{(Eq. 12-9)}$$

Where k = over-dispersion parameter = k_1/L (km), where k_1 = 0.38. The coefficient k_1 is provided by the HSM for two-lane rural roads, after the conversion from miles to kilometres. The conversion has been done assuming the over-dispersion parameter equal for fixed length, independently on the unit of measure. This conversion will be valid until the availability of a native function for Italy and the related over-dispersion parameter[17].

$$N_{Expected,current,KAB,period \; of \; investigation} = [0.60 \times 5.16 + (1-0.60) \times 10] \; KAB \; crashes = 7.08 \; KAB \; crashes \qquad \text{(Eq. 12-10)}$$

[17] These elements are available for the Puglia region thanks to research conducted by the authors of this book and cited through the chapters. The authors calculated the over-dispersion parameter for the Puglia region: it is equal to 0.81/L (km). Using this value instead of the one provided by the HSM, the result slightly changes.

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$$N_{Expected,current,period\ of\ investigation} = N_{Expected,current,KAB,period\ of\ investigation}\ x \left(\frac{100}{17.6}\right) = 40.20\ crashes \qquad \text{(Eq. 12-11)}$$

$$N_{Expected,current} = \frac{N_{Expected,current,period\ of\ investigation}}{period\ of\ investigation} = 8.04\ crashes/year \qquad \text{(Eq. 12-12)}$$

In the previous equation, the proportion of KAB crashes was used again to convert them into total expected crashes.

$$N_{Expected,intervention\ 1} = N_{Expected,current}\ x\ CMF_{itntervention\ 1} = 8.04\ crashes\ x\ 0.82 = 6.59\ crashes \qquad \text{(Eq. 12-13)}$$

$$\Delta N_{Expected,intervention\ 1,lifetime} = (N_{Expected,current} - N_{Expected,intervention\ 1})\ x\ Lifetime = (8.04 - 6.59)\ \frac{crashes}{year}\ x\ 10\ years = 14.47\ crashes \qquad \text{(Eq. 12-14)}$$

$$N_{Expected,intervention\ 2} = N_{Expected,current}\ x\ CMF_{itntervention\ 2} = 8.04\ \frac{crashes}{year}\ x\ 0.80 = 6.43\ \frac{crashes}{year} \qquad \text{(Eq. 12-15)}$$

$$\Delta N_{Expected,intervention\ 2,lifetime} = (N_{Expected,current} - N_{Expected,intervention\ 2})\ x\ Lifetime = (8.04 - 6.43)\ \frac{crashes}{year}\ x\ 10 = 16.08\ crashes \qquad \text{(Eq. 12-16)}$$

Assuming that the two interventions have the same cost and the same lifetime (and neglecting the cost actualization), the relevant agency would surely choose the intervention 2, since it may have a greater benefit, leading to an expected crash reduction of 18.14 over 10 years, instead of 16.32, reduction provided by the intervention 1.

Instead of using the regional coefficient calculated for Puglia (assuming it as unavailable), the results obtained by using the overall Italian coefficients are the following:

- $\Delta N_{Expected,intervention\ 1} = 15.80\ crashes$
- $\Delta N_{Expected,intervention\ 2} = 17.55\ crashes$

In order to choose one of the two countermeasures, the less precise calibration coefficient would have not affected the choice itself. This is because both the expected benefits would have increased, even though due to the greater coefficient for low traffic volumes (the overall Italian coefficient is 1.44, rather than the regional coefficient: 1.24).

Assuming for example the same characteristics of the segments, but placed in two different regions, one in Puglia (Cc = 1.24 for low traffic volumes) and the other in Veneto (Cc = 1.71 for low traffic volumes), the hypothesis is to implement the intervention 1 in Veneto and the intervention 2 in Puglia. The priority choice between the two sites is conducted at the national level. The same procedure used in the previous case is used here as well:

- $\Delta N_{Expected,intervention\ 1,Veneto} = 17.36\ crashes$
- $\Delta N_{Expected,intervention\ 2,Puglia} = 16.08\ crashes$

The priority is given to the site in Veneto because it leads to greater benefits. However, whether the overall Italian calibration coefficient (1.44) is used for both sites, the results are shown below:

- $\Delta N_{Expected,intervention\ 1,Veneto} = 15.80\ crashes$
- $\Delta N_{Expected,intervention\ 2,Puglia} = 17.55\ crashes$

In this case, the priority is given to Puglia. This example shows blatantly the importance of a reliable calibration coefficient to take accurate decisions about priority of interventions at the national level (between different regions, macro-regions or sites with different terrain elevations and traffic), macro-regional and regional levels (between sites with different traffic and terrain conditions).

The reliability of the calibration coefficient has an important role in complex evaluations about the crash reduction rate, for example in the benefit-cost analysis.

12.3 General remarks about the estimation of a local SPF

As mentioned in the section 12.1, in case of availability of reliable data and feasibility of their development, the use of local SPFs is highly recommended with respect to existing SPFs (AASHTO, 2010[4]). In fact, they can take into account all the variables and road/traffic-related characteristics which are crash predictors, within a single model to predict crash frequencies.

One big concern is indeed related to the computational burden that a local SPF implies compared to the calibration procedure. In many cases, this effort could not lead to remarkable differences in the precision and reliability of results. This is true especially in cases of lack of data to be processed and modelled. HSM requires at least 3 years of data to be collected in order to rely on a good dataset to perform the estimation of a novel SPF model.

When there are low crash frequencies for the studied segments, negative binomial crash prediction models can be used (Lord et al. 2006[14]) and more data are required to validate the developed model.

However, as already mentioned, it is impossible to know a-priori whether the local SPF outperform the calibration of an existing SPF, so the choice depends on the data availability and on the experience of the road safety practitioners/researchers, who can select the best alternative for the analysed case.

Developing a LF (Local Function) requires a greater effort because it is related to local road sites and the variables depend on specific case and conditions, so it is based on a wide dataset and data collection procedure. In North America, some cases of local SPFs already exist, including specific variables (e.g., trucks traffic percentage[18] or separate predictions for curves and road tangents[19]), which are considered because of their influence on crashes in those specific cases.

Moreover, the choice of the SPF functional form may also be based on the best fitting model. For example, Farid et al. (2019)[20] tested several possible different SPF modelling techniques, by assessing their outcomes and advantages in different conditions.

An extended review of possible alternative methods for modelling crash frequency data, together with their assessment, was provided by Lord and Mannering (2010)[21].

In Europe, the development of local SPFs is still in progress; so, in this chapter, the most important studies leading to the development of local SPFs, according to the HSM, for rural and urban road segments are shown, focusing particularly on the Italian case.

12.3.1 Estimation of local SPFs for Italy: rural roads

Among the studies conducted to develop a local SPFs for Italian rural roads, most of them are related to two-lane rural roads[5,22,23] (even if multilane rural roads were deeply examined too, considering also motorways[24,25]). Note that, in Europe, the road crash fatalities occurrence on two-lane rural roads is approximately 60% of the total road crash fatalities[22].

For example, the study by Cafiso et al. (2010)[22], starting from an extensive data collection and modelling effort, reports about a local SPF for crash predictions on two-lane rural roads. In this research, the SPF was

[18] Brimley B., Saito M., Schultz G. (2012), "Calibration of Highway Safety Manual safety performance function: development of new models for rural two-lane two-way highways", *Transportation Research Record: Journal of the Transportation Research Board* 2279, 82-89.

[19] Gooch J.P., Gayah V.V., Donnell E.T. (2018), "Safety performance functions for horizontal curves and tangents on two lanes, two-way rural roads", *Accident Analysis & Prevention* 120, 28-37.

[20] Farid A., Abdel-Aty M., Lee J. (2019), "Comparative analysis of multiple techniques for developing and transferring safety performance functions", *Accident Analysis & Prevention* 122, 85-98.

[21] Lord D., Mannering F. (2010), "The statistical analysis of crash-frequency data: a review and assessment of methodological alternatives", *Transporation Research Part A: Policy and Practice* 44 (5), 291-305.

[22] Cafiso S., Di Graziano A., Di Silvestro G., La Cava G., Persaud B. (2010), "Development of comprehensive accident models for two-lane rural highways using exposure, geometry, consistency and context variables", *Accident Analysis & Prevention,* 42 (4), 1072-1079.

[23] Russo F., Busiello M., Dell'Acqua G. (2016), "Safety performance functions for crash severity on undivided rural roads". *Accident Analysis & Prevention* 93, 75-91.

[24] Montella A., Colantuoni L., Lamberti R. (2008), "Crash prediction models for rural motorways". *Transportation Research Record: Journal of the Transportation Research Board* 2083(1), 180-189.

[25] Caliendo C., Guida M., Parisi A. (2007), "A crash-prediction model for multilane roads". *Accident Analysis & Prevention* 39(4), 657-670.

defined by using a unique combination of exposure, geometry, consistency and context variables directly related to the safety performance. The data were collected by cinematic GPS surveys and road safety inspections. Three negative binomial models were targeted as best fit models at the end of the modelling process. The first one included only the road segment length and traffic volume as predictor variables. However, exposure, geometry, consistency and context factors variables were included in the other two models. Despite of the acceptable goodness of fit obtained, such models could be used together with results from other studies in order to estimate the relevant CMFs and for safety assessments[22].

Russo et al. (2016)[23], instead focused their research on the development of local SPFs to predict the fatal/injury crash frequency on two-lane rural road segments The developed SPFs relate the crash occurrence and the crash severity, based on a sample of 2000 km of investigated roads of which the annual average daily traffic, lane width, curvature change rate, length, and vertical grade were collected. Four negative binomial regression models were finally developed in order to predict injury crash frequency or injury and fatalities per year for different sub-sets (non-fatal injuries occurred, at least one fatality occurred together with one injury) and the final expected frequency of injuries and fatalities per year. An analysis based on the residuals confirmed the effectiveness of the developed SPFs.

Intini et al., 2019[5] considered the geographic variability in the SPF estimation together with the road geometric and traffic variables. The assumptions and datasets were already explained in the previous sections, while presenting the calibration of existing SPFs, considering the regional variability. Crash data were retrieved from the Italian National Institute of Statistics (ISTAT)-Italian Automobile Club (ACI). The variables chosen for the local development of the SPF were not the same used for the calibration of existing SPFs (Colonna et al., 2016[6]: AADT, length of road segments, road width, shoulder type, radius of curvature, two-way left-turn lanes). In fact, some variables were not considered reliable in this context because of the methodology of detection. So, the variables which were used in this case were: AADT, segment length, road width, shoulder type, radius of curvature, curve ratio, slope, driveway density, region and elevation. The road sections (between two major intersections or significant cross-sectional changes) included in the database have a significant and then they were divided into homogenous parts, where the main geometric variables are constant.

Moreover, the left shoulder width was aggregated with the road width in a unique variable “road width”[20] since they are strongly inter-related. Shoulder types were classified into: paved, gravel, composite and turf. The curve ratio and radius of curvature were aggregated in only one variable, MC, which is the weighted mean of the segment curvature (1/km). The used variables are similar to those required by the HSM for the SPF estimation.

A Negative Binomial (NB) distribution of the errors and a natural logarithmic link function[26,27] were used according to previous literature. The model form used for the regression procedure is described in the following equation:

$$E(Y) = \exp(\beta_0) * L^{\beta_1} * AADT^{\beta_2} * \exp\left(\sum_{i=3}^{n} \beta_i X_i\right) \quad \text{(Eq.12-17)}$$

Where:

- E(Y) = predicted number of fatal/injury crashes per year (crashes/year);
- L = length of the segment (m);
- $\beta_0, \beta_2, \dots, \beta_n$ = estimated coefficients of the regression (β_1 is set to 1);
- $X_3, X_4, \dots, X_n$ = regression variables considered, other than segment length and AADT: road width, shoulder type, radius of curvature, curve ratio, slope, driveway density, Roadside Hazard Rating (RHR), region, elevation.

Among all the possible models obtainable, a model was selected based on statistical and practical considerations.

[26] Hilbe J.M. (2011), *Negative Binomial Regression*, Cambridge University Press, Cambridge, UK.
[27] Chatterjee S., Simonoff J.S. (2013), *Handbook of Regression Analysis*, John Wiley & Sons, New Jersey, USA.

Tab. 12.4: Estimated effects for the selected Italian SPF model (based on Intini et al., 2019[5]).

Variables	*Type and modalities*	*Effects of the selected variables**
AADT [vehicles/day]	Continuous	++
Segment length [m]	Continuous	(fixed to 1)[14]
Shoulder type [-]	Categorical: 0 – Paved 1 – Mixed-Composite/Gravel 2 – Turf	 + +
Weighted mean curvature [1/km]	Continuous	+

*++ = parameter estimate > 1, + = parameter estimate included between 0 and 1.

The variables which resulted as significant predictors for the crash occurrence were: AADT, shoulder type, weighted curvature. AADT is positively related to crashes (indicating a more than linear relationship). Paved shoulder seems to be safer than other materials. Curvature is positively related to crashes (the more curved are the segments, the more the crashes increase). In this specific study, a comparison between the local SPF and the transferred SPF was made. In summary, even if the prediction capabilities of the locally derived SPF are greater than those of the calibrated SPF, the differences were found to be not statistically significant. This means that, in this specific case, the effort of developing a novel SPF, based on the same sample which can be used for HSM calibration, cannot be justified by a significant prediction improvement.

12.3.2 Estimation of local SPFs for Italy: urban roads

Contrary to the rural case, analyses in the urban context are not frequently found in the European literature, while several studies have been developed in North America. Among the most relevant European studies, the studies by Gomes et al. (2012)[28] and Greibe et al. (2003)[29] that have dealt with the development of local SPFs both for intersections and urban road segments, are presented here. In Italy, some authors (see e.g.,[30,31]) have also developed urban local SPF models. A summary of the considered models for urban road segments and intersections is reported in the following Table 12.5. It includes the variables that have been maintained in the final statistically acceptable models presented by the authors and some additional variables that could be interesting for road safety modelling, but that have not been found in any model considered. A comparison[32] with the aforementioned European models can be also made by looking at values in Table 12.5. The Italian models have few analysed variables for segments than the European models mentioned above.

[28] Gomes S.V., Geedipally S.R., Lord D. (2012). "Estimating the safety performance of urban intersections in Lisbon, Portugal". *Safety Science* 50, 1732-1739.

[29] Greibe P. (2003), "Crash prediction models for urban roads". *Accident Analysis and Prevention* 35, 273-285.

[30] Fancello G., Soddu S., Fadda P. (2018), "A crash prediction model for urban road networks". *Journal of Transportation Safety and Security* 10, 387-405.

[31] Canale S., Leonardi S., Pappalardo G. (2005), "The reliability of the urban road network: Crash forecast models". *Proceedings of the 3rd International SIIV Conference*, Bari, Italy, 22-24 September; 1-22.

[32] Colonna, P., Intini P., Berloco, N., Fedele V., Masi G., Ranieri V. (2019), "An Integrated Design Framework For Safety interventions on Existing Urban Roads - Development and Case Study Application", *Safety*, 5, 1-13.

Tab. 12.5: Variables included in some European models retrieved in literature for road segments and intersections, with indications of some additional important variables not retrieved in any model (based on Colonna et al., 2019[32]).

Segments				Intersections								
Variables	*Included in final models as retrieved in*:*			Variables	*Included in final models as retrieved in*:*							
	29	30	any		28 Three-*Legged Intersections*	31**	30^	any	28 Four-*Legged Intersections*	31**	31^	any
AADT	**X**	**X**		AADT of major road section	**X**	**X**			**X**	**X** (no-control, stop)		
Section length	**X**			AADT of minor road section	**X**	**X** (stop)			**X**	**X**		
Speed limit	**X**			Total entering AADT			**X**				**X**	
Road width	**X**			Lane balance	**X**				**X**			
Number of accesses	**X**	**X**		Median on one major leg	**X**							
Number of minor exits	**X**			Median on two major legs	**X**	**X** (stop)						
Parking	**X**			Median on two minor legs	**X**							
Land use	**X**			Total entering lanes (major road)					**X**			
One-way			**X**	Number of lanes (minor road)						**X** (signalized)		
Number of lanes			**X**	Average lane width (minor road)		**X** (stop)			**X**	**X** (no-control, signalized)		
Road signs on minor roads/accesses			**X**	Number of one-way legs					**X**			
Pavement conditions			**X**	One-way on major road		**X** (no-control)				**X** (no-control, signalized)		
Road markings			**X**	One-way on minor road		**X** (stop)				**X** (no-control)		
Presence of bike lanes/paths			**X**	Right turn on major road^^	**X**				**X**	**X** (stop, signalized)		
Sidewalk width			**X**	Right turn on minor road^^						**X** (stop, signalized)		
Median presence			**X**	Left turn on major road^^		**X** (no-control)				**X** (stop)		
Bus stops			**X**	Left turn on minor road^^						**X** (stop, signalized)		
Bus–taxi lane			**X**	Sidewalk width on major road		**X** (stop)				**X** (signalized)		
				Sidewalk width on minor road						**X** (no-control)		
				Grade on major road section°		**X** (no-control)				**X** (stop, signalized)		
				Grade on minor road section°		**X**				**X** (no-control, stop)		
				Road markings		**X**						
				Phasing of signals						**X** (signalized)		
				Sight distance				**X**				**X**
				Pavement conditions				**X**				**X**
				Presence of bike lanes/paths				**X**				**X**
				Bus–taxi lane				**X**				**X**

*The number below is the reference number of the study in this chapter, while "any" stands for variable not found in any model.
**Different SPFs for each intersection category (no-control three-leg and four-leg intersections, stop-controlled three-leg and four-leg intersections, and signalised four-leg intersections), also considering different crash types (here not reported).
^There is a unique SPF for all intersections, including different numbers of legs.
^^Differentiated in protected and permitted left/right-turn lane in the data collection by Canale et al., 2005[31].
°Differentiated in steep and level grades in the data collection by Canale et al., 2005[31].

Moreover, some local SPFs for urban areas were developed by Intini et al (2020)[33]. In this case, the City of Bari (Italy) was used as a field for data collection, given that the study was conducted within the Pa.S.S.S. (Scientific Park for Road Safety) National research project (Italian Ministry of Transport and Infrastructures/City of Bari). Relevant characteristics of the roads in the Bari urban area were collected in a database to develop local Safety Performance Functions (SPFs).

The typical equation structure is shown below for both segments and intersections:

$$N_{SPF,segments} = e^{\beta 0,S} * AADT^{\beta 1,S} * L^{\beta 2,S} * e^{\sum_{i=3}^{n} \beta i,S\, Xi,S} \quad \text{(Eq. 12-18)}$$

$$N_{SPF,intersections} = e^{\beta 0,I} * (AADT_{maj} + AADT_{min})^{\beta 1,I} * e^{\sum_{i=2}^{n} \beta i,I\, Xi,I} \quad \text{(Eq. 12-19)}$$

Where:

- AADT = Annual Average Daily Traffic for segments;
- $AADT_{maj}$ = AADT for the major road at intersection (carrying the highest amount of traffic);
- $AADT_{min}$ = AADT for the minor road at intersection (carrying the lowest amount of traffic);
- L = segment length (m);
- $X_{i,S}$ = other predictors for segments (numerical or categorical);
- $X_{i,I}$ = other predictors for intersections (numerical or categorical);
- $\beta_{i,S}$ = estimate of the coefficients associated to each crash predictor for segments through maximum likelihood estimation ($\beta_{0,S}$ is the estimate for the intercept);
- $\beta_{i,I}$ = estimate of the coefficients associated to each crash predictor for intersections through maximum likelihood estimation ($\beta_{0,I}$ is the estimate for the intercept).

Data were collected from 2012 to 2016 and 320 homogenous road segments for a total of 43 km were investigated. The same temporal window was used for the intersections and 122 intersections were studied. The variables used are described in the following table.

A negative binomial regression has been used, together with the stepwise procedure. The modelling process has led to 8 SPFs, differentiated into subsets of:

- all segments,
- one-way segments,
- two-way segments,
- all intersections,
- unsignalized intersections,
- signalized intersections,
- 3-legs intersections,
- 4-legs intersections.

Note that most of the obtained results are self-explaining, while some of them are difficult to be interpreted. More details on the interpretation of the variables are provided in the study, as well as the comparison with models found in literature.

The effect of pavements/marking maintenance, critical sight distance at intersections, vertical signs on driveways/minor roads and cycle path crossings are demonstrated and they may be considered for further safety predictions.

Moreover, this study provides some insights on the issue of dividing intersections and segments into subsets and on additional predictors to be considered for the estimation of urban SPFs. However, it is based on a limited number of segments and intersections, which may negatively influence the crash predictions[34]. Hence, more data are required to validate the developed models.

[33] Intini P., Berloco N., Cavalluzzi G., Lord D., Ranieri V., Colonna P. (2020), "The variability of Urban Safety Performance Functions for Different Road Elements: an Italian Case Study", Paper presented at TRB 99th Annual Meeting - January 12-16, Paper Number: 20-04921. Paper submitted for journal publication.

[34] Lord, D. (2006), "Modeling motor vehicle crashes using Poisson-gamma models: Examining the effects of low sample mean values and small sample size on the estimation of the fixed dispersion parameter". *Accident Analysis & Prevention,* 38(4), 751-766.

Tab. 12.6: Predictors for segments and intersections (based on Intini et al., 2020[33]). For predictors in italics, no statistically significant parameter was estimated in any of the selected models for segments and intersections.

Variables for segments	*Modalities*	*Variables for intersections*	*Modalities*
AADT	Numerical	Total AADT	Numerical
Length (m)	Numerical	Main AADT/Total AADT	Numerical
Speed Limit = 50 km/h	0 – Yes 1 – No (> 50 km/h)	*Lane balance*	0 – No 1 – Yes
Road width (m)	Numerical	Median on the main road	0 – No 1 – Yes
Type of lanes	0 – 1 lane 1 – > 1 lane (up to 3) 2 – 1+1 lane 3 – > 1+1 lanes (up to 3+3)	*Median on the secondary road*	0 – No 1 – Yes
Minor roads/driveways per km	Numerical	Entering lanes (main road)	Numerical
Vertical signs on minor roads/driveways	0 – No 1 – Yes	*Mean lane width at intersections (m)*	Numerical
Bad maintenance of pavements	0 – No 1 – Yes	Critical sight distance (m)	Numerical
Road markings	0 – Absent or illegible 1 – Illegible 2 – Yes	*One-way legs*	Numerical
Parking type	0 – Prohibited 1 – At one side 2 – At both sides 3 – Mixed	Specialized turning lane	0 – No 1 – Yes
Cycle paths	0 – No 1 – Yes	Traffic control	0 – No 1 – Give-way/Stop 2 – Traffic lights 3 – Traffic lights + dedicated turning
Sidewalks	0 – No 1 – Yes	Bad maintenance of pavements	0 – No 1 – Yes
Median	0 – No 1 – Yes	Cycle path crossing	0 – No 1 – Yes
Bus stop	0 – No 1 – Yes	Sidewalks	0 – No 1 – Yes (both sides)
Bus/taxi lane	0 – No 1 – Yes	Bus stops	0 – No 1 – Yes
Land use	0 – Residential 1 – Mainly commercial 2 – Other	*Bus/taxi lanes on intersecting roads*	0 – No 1 – Yes

12.4 Example of application of the HSM method in Scotland

The application of the calibration SPF procedure as well as the estimation of a preliminary local SPF for Scottish rural roads has been investigated by Intini et al. (2019)[5], similarly to the study conducted for the Italian rural road case, previously shown.

The study was run starting from two different datasets: one for the Average Annual Daily Traffic (AADT), taken from the UK Department for Transport, and another for the crashes (fatal + injury), from the online portal https://data.gov.uk/. These datasets regarded secondary two-lane rural roads. The data for traffic were divided according to the threshold of 2000 vehicles/day (low and high traffic), since the traffic volumes were generally low in Scotland. After, these data were assigned to the road sections, intended as the parts of the 66 segments between two main intersections investigated. Crash data were related to the period 2012-2014 (at least 3 years[1]), leading to 101 crashes in total. A sufficient number of zero-count sites were collected to account for the estimated low mean crashes/km rate in the part of network studied.

As stated for the corresponding Italian case, the analysis may not be reliable considering the SPF calibration and estimation for the entire Scotland, so Scotland was divided into 2 parts: Lowlands and Highlands. Apart from these distinctions, the terrain elevation variability was not considered since the overall distribution of elevations was far below 400 m.

12.4.1 Calibration of SPFs: Scottish rural roads

The calibration factor, Cc and the coefficient of variation, c_v have been calculated (Eq. 12-2 and Eq. 12-3). The table below shows the calibration results.

The results show that a regional effect between the two macro-regions cannot be found, as highlighted by the similar overall calibration factors for both Highlands and Lowlands.

The calibration looks reliable for most of the data, except for the Highlands, when there is a distinction based on traffic volume, and for Lowlands in presence of high traffic volume. The calibration factor (Cc < 1) indicates that the baseline SPF may overestimate crash frequencies, especially for high traffic volumes.

Tab. 12.7: Calibration coefficient values for different regions and traffic conditions (based on Intini et al., 2019[5]).

Variable: Region	*AADT Ranges*	*Cc**
Overall	Overall	0.71
	< 2,000	1.20
	≥ 2,000	0.48
"Lowlands"	Overall	0.75
	< 2,000	1.23
	≥ 2,000	*0.41*
"Highlands"	Overall	0.66
	< 2,000	*1.11*
	≥ 2,000	*0.54*

*Cc coefficients in italics are deemed less reliable due to either related number of segments < 30 or cv [Cc] ≥ 0.20.

12.4.2 Estimation of local SPFs: Scottish rural roads

The same procedure stated for the Italian case of the estimation of a local SPF has been proposed here. The variables considered for the purposes of the study in the two macro-regions previously defined are the following: length of segments, AADT, road width, shoulder type, mean curvature, slope, driveway density, elevation and Roadside Hazard Rating (RHR). Road width includes both the width of shoulders and lanes in a synthetic variable, instead the shoulder type was classified into paved/mixed/composite shoulder, and turf shoulder.

Also, in this case, a model was selected based on statistical and practical considerations.

Tab. 12.8: Estimated effects for the selected Scottish SPF model (based on Intini et al., 20195[5]).

Variables	*Type and modalities*	*Effects of the selected variables**
Segment length [m]	Continuous	(fixed to 1)[14]
Shoulder type [-]	Categorical: 0 – Paved/Mixed/Composite 1 – Turf	-
Weighted mean curvature [1/km]	Continuous	+

*+ = parameter estimate included between 0 and 1, - = parameter estimate included between 0 and -1.

The fundamental variables which have an effect on crashes according to the developed SPF are: the mean weighted curvature and the shoulder type. The mean curvature leads to an increase in crashes, instead the turf shoulder is related to the crashes decreasing.

The traffic volume was not identified as a crash predictor in this case, probably because traffic volumes in the dataset are all low, even though this result might be analysed in further detail.

12.5 References

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13. Example of design application: rural roads

13.1 Introduction

This chapter describes an application of the concepts introduced in the previous chapters. In detail, it aims at providing a practical guide to present/future professional engineers on how to design safety interventions on existing rural roads. This chapter has then been structured as a descriptive technical report, supporting designers and public authorities in the safety enhancement of rural roads, and for educational purposes. Whereas, the case of urban roads is considered in the next chapter, by following a framework for the design application similar to that used in this chapter for rural roads.

In particular, the proposed protocol for road safety interventions is here applied to a two-way two-lane rural road segment. The segment selected for showing the application of the method in the rural environment is not provided with intersections with roads of a similar importance. However, in order to provide some insights on the solution of intersection-related issues on these types of roads, some examples are reported in the last paragraph of this chapter.

This example is related to the Italian case, and so it is referred to Italian standards and regulations. However, besides specific standards and regulations, the presented framework is independent on the particular country, considering also that the basic concepts of International standards for road design are largely similar. Hence, the general framework for road safety interventions, which includes both quantitative concepts for estimating traffic crashes (mainly taken from the HSM manual, 2010[1]) and qualitative concepts for assessing safety conditions (mainly referred to EU guidelines) could potentially be applied everywhere and integrated with local standards.

However, the reference legislation applied in the case example presented in this chapter is described below.

13.2 Reference legislation

The main Italian reference regulation for the analysis of safety conditions and design of road safety interventions is the Legislative Decree 35 of 15/03/2011 "*Implementation of Directive 2008/96 / EC on the safety management of infrastructure*" and the "*Guidelines for the safety management of road infrastructures*", published by the Ministry of Infrastructures and Transport in 2012, according to the article 8 included in the above cited decree.

The Guidelines establish the "*criteria and procedures for road safety checks on projects, the safety inspections of existing infrastructures and for the implementation of the process for classifying the safety of road networks*". They are also aimed at guiding, coordinating and standardizing the activities of all those involved in the safety of road infrastructures, including the local authorities, the relevant road agencies, managers and road safety experts, namely the project controllers and inspectors of existing roads.

In addition to use of the "*Guidelines for the safety management of road infrastructures*", the American manual "Highway Safety Manual (HSM) - 1st edition" was used as a reference, which outlines the techniques currently available for measuring, estimating and evaluating crash measures related to a road infrastructure. In particular, the HSM manual, 2010[1] was used for most of the analyses related to the design road safety interventions, since the 2012 Guidelines do not identify detailed quantitative methods for measuring and predicting crashes, based on statistical or scientific data, whilst underlining other aspects such as inspections.

As regards the geometrical and functional checks of road elements, the Decree of 5 November 2001 n. 6792 (Official Gazette S.O. # 5 to # 3. of 04/01/02) "*Functional and geometric standards for the construction of the roads*" was considered.

[1] AASHTO (2010), *Highway Safety Manual, First Edition*, Transportation Research Board, National Research Council, Washington D.C., USA.

13.3 General background for the application

The road segment selected for the application example is part of the "S.P. 239" road (ex S.S. 604), managed by the Province of Bari. It is located about 45 km south of the city of Bari and it is approximately 2 km long (from km 6+000 to km 8+000).

The identified segment falls within the administrative boundaries of the Municipality of Noci on the border with the Municipality of Gioia del Colle, as shown in Figure 13.1. The road under investigation has been selected as an application example because, during the last decade, it has been affected by several road crashes (also with fatal consequences).

According to the Safety Performance Function (SPF) for two-way two-lane rural roads calibrated for the Apulian context[2], for each average daily traffic value, a predicted number of crashes per kilometre and per year which is typical of the considered road type can be assigned.

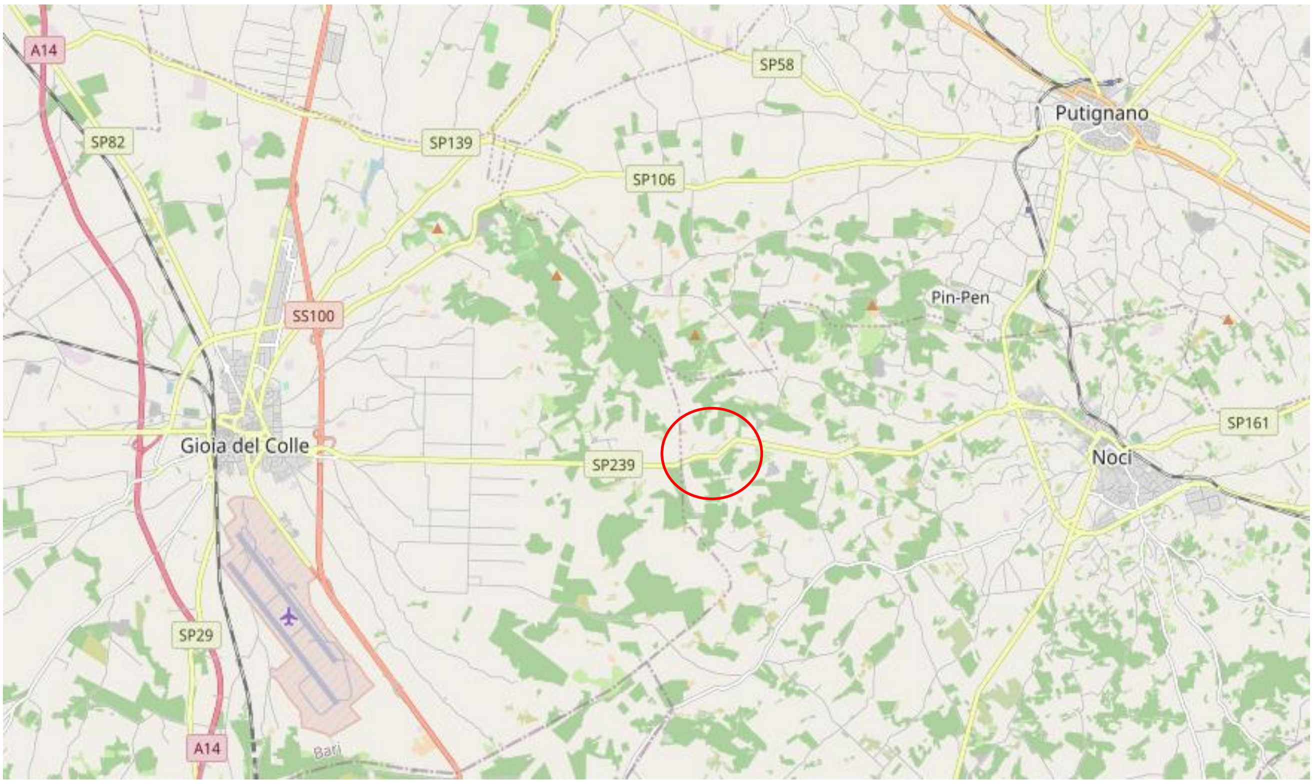

Fig. 13.1: Study Area Framework (photo source Open Street Map).

The basic SPF is provided by the Highway Safety Manual (2010)[1]. However, a value of 1.26 has been determined for the calibration coefficient Cc in the Apulian context.

$$N_{SPF} = constant\ x\ AADT \times L \times 365 \times 10^{-6} \times Cc \qquad \text{(Eq. 13-1)}$$

For the site examined, with an average daily traffic of 4202 vehicles/day, the predicted total crashes from the SPF calibrated for the Puglia region is estimated as equal to 0.88 crashes/year/km (Np in figure 13.3). In order to obtain a risk measure of the site, it is possible to compare the crashes occurred on the site (observed number of crashes) with the crashes predicted by the SPF. In this way, the safety conditions of the site can be compared with similar sites in terms of geometrical, functional and geographical characteristics (in this case data refer to the Puglia region). Therefore, it is defined in a quantitative way.

However, if native SPFs are not available or local calibration coefficients are not known (for calibration

[2] Colonna P., Berloco N., Intini P., Perruccio A., Ranieri V., Vitucci V. (2016), "Variability of the Calibration Factors of the HSM Safety Performance Functions with Traffic, Region and Terrain. The case of the Italian rural two-way undivided road network", Compendium of Papers of the 95th Annual Meeting of the Transportation Research Board, Washington D. C., USA.

coefficients referring to other Italian regions in different contexts see Colonna et al., 2016[2]), other performance metrics can be used to determine the risk measure of a site. In particular, the most easily available data are crash frequencies (referring to crashes with deaths and injuries) per year and per kilometre. Using this type of metric, and imposing 1 crash with deaths and injuries per kilometre per year as a threshold value, the selected site exceeds the threshold value for the km 7+000 - 7+999 (1.28 fatal and injury crashes per kilometre per year; an analysis of the crash data for the segment will be later discussed). In this chapter, it is possible to use the Levels of Service of Safety (LOSS) method proposed by Kononov et al. (2003)[3] and indicated in the HSM manual (2010)[1] as one of the possible road safety performance metrics, thanks to available data.

According to the original version of this method, it is possible to classify the LOSS thanks to a SPF obtained for similar sites. The possible LOSS are four and they are delimited as in Figure 13.2. Sites for which crashes per year per kilometre are observed to be higher than the predicted value by the SPF for that traffic value, may be classified as having LOSS III or IV, depending on whether the value exceeds (or not) 1.5σ from the average SPF prediction.

Such sites (in particular those for which level IV is found) are those to be considered as the most dangerous and therefore with the greatest potential for improvement from a safety point of view. However, the use of a fixed distance (corresponding to 1.5 standard deviations) from the average SPF may be affected by some problems (see chapter 10, and Kononov et al., 2015[4]).

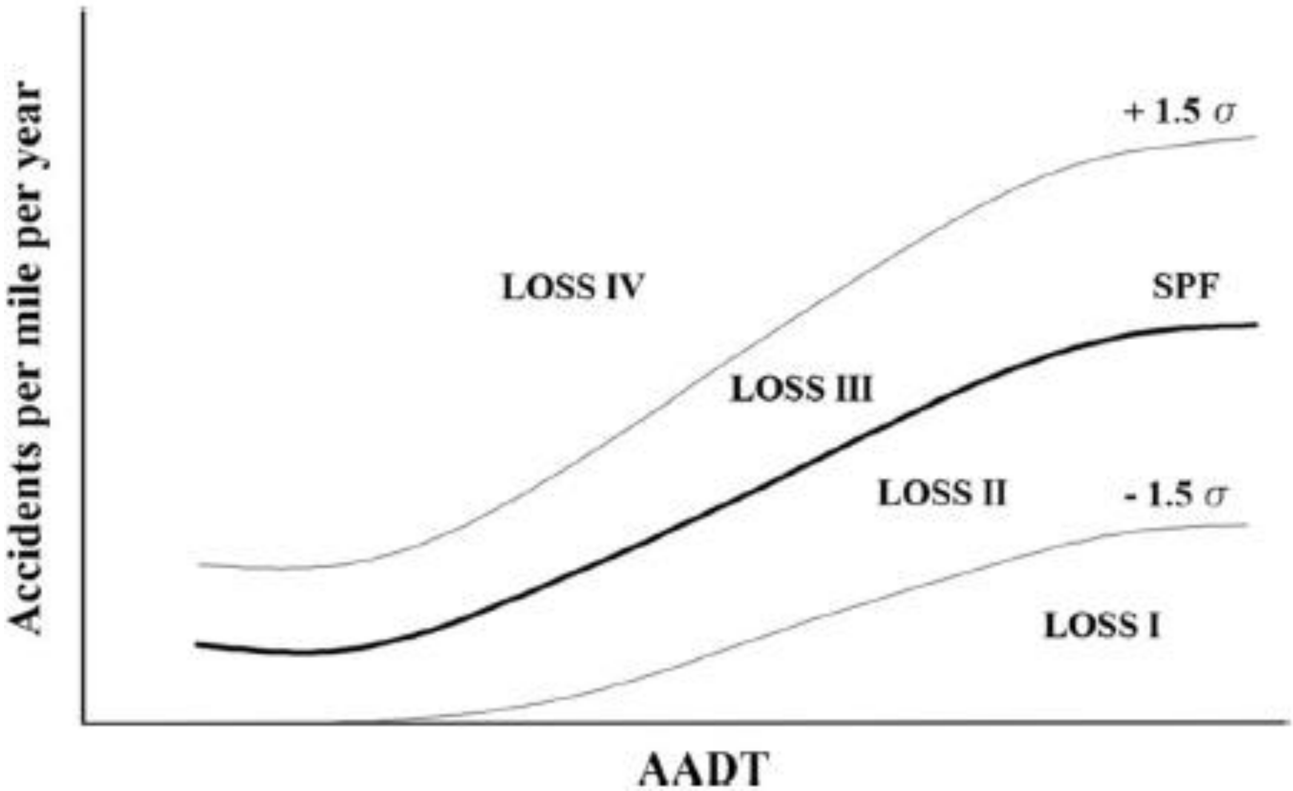

Fig. 13.2: The four Levels of Service of Safety, LOSS (based on Kononov et al., 2003[3]).

Figure 13.3 shows the application of the method described above for the identification of the safety service levels for the region Puglia, referring to the SPF for two-way two-lane rural roads regionally calibrated.

[3] Kononov J., Allery B. (2003), "Level of Service of Safety. Conceptual Blueprint and Analytical Framework", *Transportation Research Record, Journal of the Transportation Research Board,* 1840(1), 57-66.

[4] Kononov J., Durso C., Lyon C., Allery B. (2015), "Level of service of safety revisited", *Transportation Research Record, Journal of the Transportation Research Board*, 2514(1), 10-20.

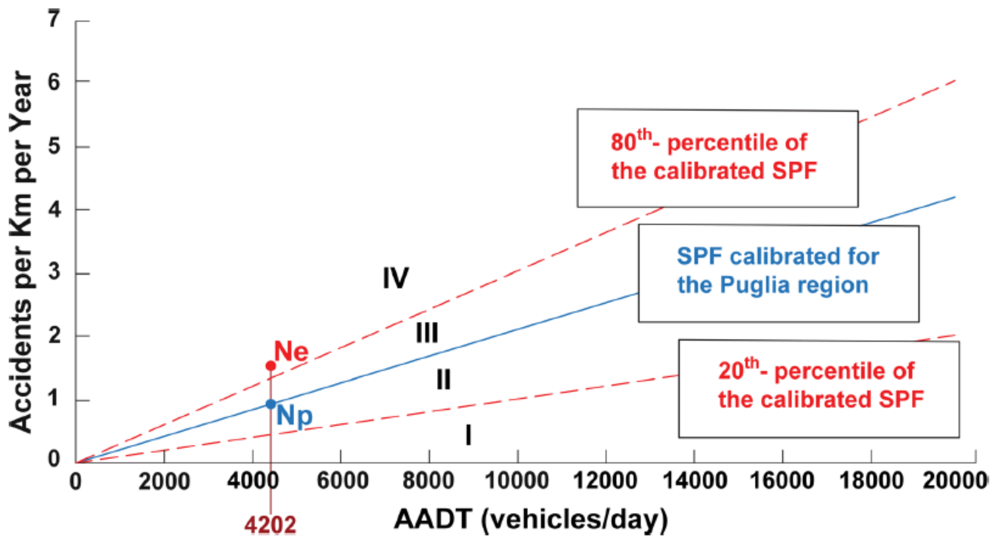

Fig. 13.3: Levels of Service of Safety for two-way two-lane rural roads in the Apulia region with indication of the number predicted by the SPF and expected number of crashes for the site under examination (Colonna et. al. 2018[5]).

For the selected site, the expected number of crashes $N_{expected}$ is calculated (corrected with the EB method, an estimate to be preferred to the simple crashes observed during the reference period, also for this type of application) and then the LOSS corresponding to the $N_{expected}$ is identified. The number of crashes is calculated using the following formula:

$$N_{Expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \qquad \text{(Eq. 13-2)}$$

$N_{predicted}$= 0.88 total crashes/year/km (from the SPF calibrated for the Puglia region) x 2 km = 1.76 crashes/year

Moreover, since, as already explained in chapter 7, in the absence of specific detailed data, it is possible to consider (in the first instance) that fatal and injury crashes account for 32.1% of the total, it can be written:

$$N_{predicted,fatal\ and\ injury} = 1.76 \frac{crashes}{year}\ x\ \left(\frac{32.1}{100}\right) = 0.57 \frac{fatal\ and\ injury\ crashes}{year}$$

$N_{Observed}$ = 11 fatal and injury crashes[6] = 1.57 fatal and injury crashes/year (over 7 years of study)

$$w = \frac{1}{1 + K\ x\ \sum N_{Predicted}} = \frac{1}{1 + \left(\frac{0{,}38}{2\ km}\right) x\ \frac{0{,}57\ Crashes\ CMF}{year}\ x\ 7\ Years} = 0.57^{7}$$

Therefore, the expected number of fatal and injury crashes for the site under investigation is, according to Eq. 13-2, equal to 0.57 x 0.57 + (1 - 0.57) x 1.57 = 1.00 fatal and injury crashes/year on the segment of 2 km and therefore 0.50 fatal and injury crashes/year/km and 0.50 x (100/32.1) = 1.55 total crashes/year/km. This value, as can be seen from Figure 13.3, falls within the LOSS-IV area (the limit between LOSS-III and LOSS-IV has been set in this case in the 80th percentile (Kononov et al., 2015[4], see the explanation about the use of percentiles in chapter 10) of the SPF forecast and corresponds to 1.27 total crashes/year/km for AADT=4202 vehicles/day).

Therefore, according to the LOSS method, the site examined would fall in the 20th percentile of the most “dangerous” sites in the Puglia region with reference to a regionally calibrated SPF. This circumstance indicates that the site has a high potential for improvement from the point of view of road safety, which is why it has been targeted for road safety interventions.

[5] Colonna P., Intini P., Berloco N., Ranieri V. (2018), “Integrated American-European protocol for safety interventions on existing two-lane rural roads”. *European transport research review*, 10(1), 5.

[6] Only fatal and injury crashes found on the segment in question in the years 2008-2014 are considered reliable and therefore this value is determined in relation to total crashes using the standard proportion proposed by the HSM. These considerations will be explained in detail in the following chapters.

[7] w = weight to be attributed to the predicted number of crashes from calibrated SPF. It was chosen to use the over-dispersion parameter provided by the HSM for this type of road, converted into the metric system: k=0.38/(L (km)), since the Apulian coefficient has been calculated.

13.4 Analysis of geometric and functional characteristics

It is possible to summarize the analysis procedure for the road segment under investigation in five phases:

- reconstruction of the horizontal and vertical alignment and identification of the geometrical elements composing it: tangents, curves having constant or variable radius (transition curves), vertical curves;
- network diagnosis: analysis of crash data, temporal and spatial reconstruction of crashes, inspection of the infrastructure with identification of critical points and deteriorations;
- verification of the layout: comparison of the values obtained from the horizontal and vertical alignment reconstruction with the Italian standards (Ministerial Decree 05/11/20019);
- analysis of possible countermeasures, in relation to the evaluation carried out in the previous phases, by studying their effectiveness and assessing the relative variation in crashes;
- economic evaluation of countermeasures, whereby it is possible to choose the countermeasure (or set of countermeasures) that gives the maximum ratio or maximum difference between benefits and costs.

13.4.1 Identification of geometric characteristics

In order to reconstruct geometric elements of the selected road segment, it is certainly recommended to carry out a detailed celerimetric survey. It will identify the road infrastructure features and the surrounding territory.

In this application example, the reconstructions were carried out by using the regional technical maps digitally available (www.sit.puglia.it). Since no complete and detailed campaign of celerimetric surveys were conducted, inspections were carried out to verify the longitudinal and transversal slopes.

The reconstruction of the layout and the graphic elaborations were carried out in CAD environment, reporting and superimposing technical and thematic maps on aerial photos of the Apulia Region.

13.4.2 Reconstruction of the horizontal alignment

The horizontal reconstruction of the road segment involves identifying the following elements:

- tangents (characterized by the length Lri);
- circular curves (characterized by the length Lci and the radius Ri);
- transition curves (characterized by the length Lai and parameter Ai).

For the identification of tangents, it is possible to draw lines along the road curbs (or along road lane markings that can be seen from the aerial photo). In this way it is possible to identify points where they deviate significantly from curbs (or road markings), then determining the extremes of each tangent.

Afterwards, to determine the circular curves, cross sections were inserted along segments which are under investigation and circumferences were drawn for three points (the midpoints of the previously inserted cross sections), this operation is repeated several times to obtain the most realistic arc. The same operation can be carried out by taking as reference the road centreline, when it can be seen from the aerial photo.

The remaining parts of the layout (not represented by tangents and arcs) can be identified as transition curves. The transition curve is geometrically represented by the equation:

$$r \times s = A^2 \qquad \text{(Eq. 13-3)}$$

where:

- r = radius of curvature;
- s = length;
- A = scale parameter.

It is possible to obtain the parameter A of the transition curve with the available data, in the first instance. In the section where the transition from the transition curve to the circular curve takes place, the value “r” coincides with the value of the curvature radius of the circular curve; the value “s” coincides with the total transition curve length (from the tangent end to the curve beginning). Therefore, it is possible to obtain the parameter “A”:

$$A = \sqrt{R_{curve} \times S_{transition\ curve}} \qquad \text{(Eq. 13-4)}$$

By measuring these parameters, the key elements for the horizontal alignment are available.

Tab. 13.1: Summary of the horizontal alignment characteristics.

Type	*Starting point [m]*	*Ending point [m]*	*Length [m]*	*Parameter [m]*	*Starting Radius [m]*	*Ending Radius [m]*	*Direction*	*Cross slope, right [%]*	*Cross slope, left [%]*	*Speed [km/h]*
Tangent	0.0	220.9	220.9	0.0	0.0	0.0		-2.5	-2.5	100.0
Transition curve	220.9	258.5	37.6	80.0	0.0	170.0	Left	0.0	0.0	74.0
Curve	258.5	282.1	23.6	0.0	170.0	170.0	Left	6.7	-6.7	69.0
Transition curve	282.1	329.7	47.6	90.0	170.0	0.0	Left	0.0	0.0	75.0
Tangent	329.7	686.7	357.0	0.0	0.0	0.0		-2.5	-2.5	100.0
Transition curve	686.7	721.2	34.5	100.0	0.0	290.0	Right	0.0	0.0	89.0
Curve	721.2	778.3	57.1	0.0	290.0	290.0	Right	-5.6	5.6	85.0
Transition curve	778.3	855.9	77.6	150.0	290.0	0.0	Right	0.0	0.0	94.0
Tangent	855.9	1053.1	197.2	0.0	0.0	0.0		-2.5	-2.5	94.0
Transition curve	1053.1	1068.7	15.6	50.0	0.0	160.0	Left	0.0	0.0	70.0
Curve	1068.7	1163.6	95.0	0.0	160.0	160.0	Left	6.2	-6.2	68.0
Transition curve	1163.6	1179.3	15.6	50.0	160.0	0.0	Left	0.0	0.0	70.0
Tangent	1179.3	1582.3	403.1	0.0	0.0	0.0		-2.5	-2.5	91.0
Transition curve	1582.3	1613.6	31.3	50.0	0.0	80.0	Right	0.0	0.0	56.0
Curve	1613.6	1649.3	35.7	0.0	80.0	80.0	Right	-6.5	6.5	51.0
Transition curve	1649.3	1680.6	31.3	50.0	80.0	0.0	Right	0.0	0.0	56.0
Tangent	1680.6	1921.8	241.2	0.0	0.0	0.0		-2.5	-2.5	91.0

In some cases, the regional technical maps (C.T.R.) were different from the most recent aerial photos. Therefore, the regional aerial photo was considered more reliable for the reconstruction of the layout.

The selected 2 km long road segment cannot be considered as independent from the rest of the S.P. 239 layout. In fact, in order to carry out some geometrical and functional verifications of the segment, the reference context and the geometrical elements of the centreline upstream and downstream of it should be analysed.

In particular, it was observed that before Curve 1 there is a tangent 6 km long; it is interrupted by a signalized intersection about 3.70 km before the Curve 1. At the same time, the tangent following Curve 4 is 1.76 km long.

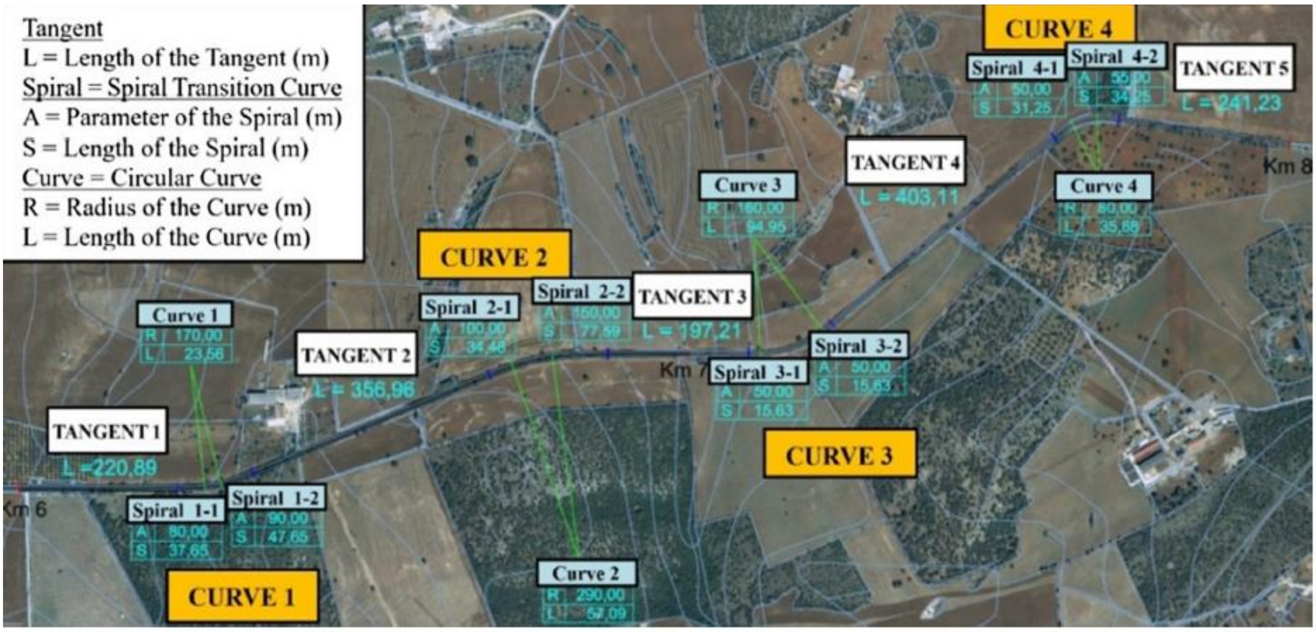

Fig. 13.4: General overview of the analysed segment with division in homogenous parts (Colonna et al., 2018[5]).

13.4.3 Reconstruction of the vertical alignment

The reconstruction of the vertical alignment is useful to identify:

- grades (characterized by the slope i);
- vertical curves (crest or sag vertical curves, characterized by the vertical radius Rv).

For the subsequent operations and verifications, a road design software is advised. In this study, the "Civil Design" software (Digicorp Ingegneria s.r.l.) has been used, which provides educational licenses to teachers and students of the Scientific Educational Sector ICAR/04 of the Politecnico di Bari University.

In order to obtain the elevation profile, a digital terrain model (D.T.M.) is created, starting from the available data from the regional database.

For the aim of reconstructing the elevation profile, if detailed surveys are not available, at least on-site inspections are needed, to identify heights and singular points. Once the field investigations have been completed, it is possible to report the longitudinal profile of the segment in the CAD environment. The reconstruction should combine results from the on-site inspections to information retrieved from the regional digital maps.

Once grades are obtained by connecting known points, the vertical curves can be drawn for linking grades between them, with the help of the CAD-based road design software. Also, in this case, information retrieved from digital maps and in-site inspections are crucial.

Tab. 13.2: Vertical characteristics of the road alignment.

Vertex

Element ID	*km*	*Height [m]*	*Slope [%]*	*Vertical grade*	*Length [m]*
0	0.0	375.2	0.0	0.0	0.0
1	110.5	372.3	-2.6	-2.9	110.6
2	393.5	386.3	4.9	14.0	283.3
3	549.0	378.5	-2.0	-7.8	155.6
4	687.1	378.4	0.0	0.0	138.2
5	1007.5	363.7	-4.6	-14.7	320.7
6	1337.0	367.6	1.2	4.0	329.6
7	1897.8	386.1	3.3	18.4	555.1

Vertical curve

Element ID	*Type*	*Vertical Radius (m)*	*Delta i (%)**	*Length*	*Starting point*	*Ending point*	*VD (km/h)***
1	Parabolic	783.9	7.5	59.0	81.0	140.0	90.9
2	Parabolic	977.2	-9.9	97.1	345.0	442.0	88.8
3	Parabolic	1246.4	5.0	61.9	518.0	579.9	100.0
4	Parabolic	2063.8	-4.6	94.3	640.0	734.3	93.9
5	Parabolic	947.5	5.8	55.0	980.0	1035.0	79.0
6	Parabolic	4606.0	2.1	97.8	1288.1	1385.9	90.9
7	Parabolic	1777.5	-0.9	16.4	1883.6	1900.0	87.8

*Delta i [%]: it expresses the gradient between the element n and the element n+1

**V_D [Km/h]: it is the design speed

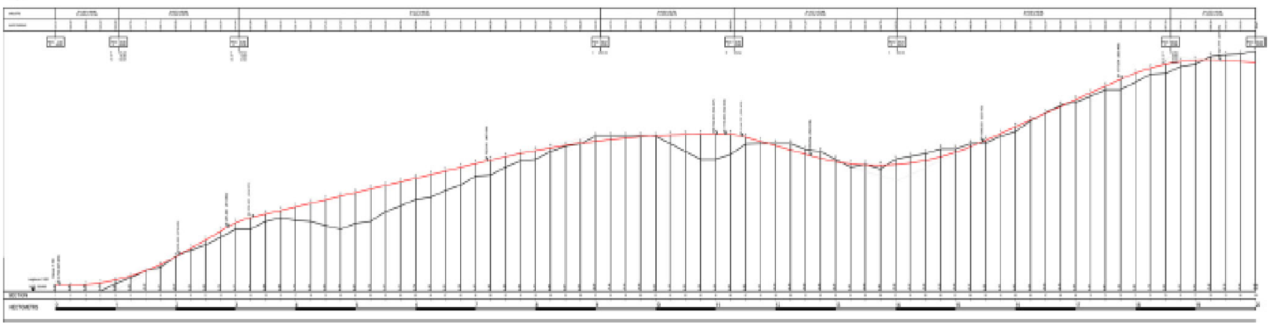

Fig. 13.5: Elevation profile (Colonna et al., 2018[5]).

13.4.4 Preliminary identification of homogeneous road segments

The analysis of the geometric characteristics of the road layout being studied leads to the division into "homogeneous road segments". For the identification of homogeneous road segments, the following elements must be taken into account:

- geometric characteristics of the layout;
- functional class of the road (and any further classifications based on different organizations of the cross section, such as the change in the number of lanes);
- context and environmental-based features (for example flat or rolling terrains);
- traffic (volumes, composition, temporal variability, etc.).

The analysis of the geometric characteristics allows a preliminary classification into "arcs", which represent road segments, and into "nodes", which represent intersections. The road arc between two nodes can consist of several homogeneous road segments. In fact, if the geometric characteristics are largely variable within the road arc (variation of the road width, presence/absence of shoulders, variation of slopes, etc.), the arc must be divided into further road homogeneous segments. In addition, the HSM (2010)[1] provides a minimum length of the homogeneous segment equal to 1/10 of a mile (about 160 m). For this reason, the first homogeneous segment, not respecting the minimum length requirement, was enlarged by about 20 meters before the 6.000 km milepost (figure below). The division is shown in the figure 13.4.

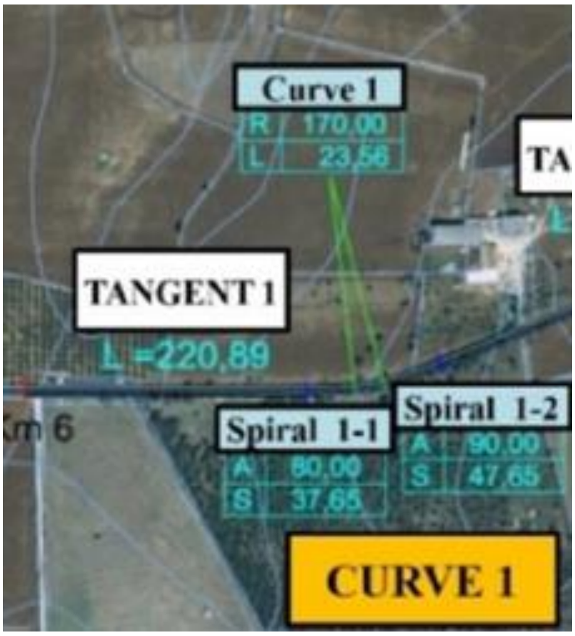

Fig. 13.6: Detail of the curve 1 (Colonna et al., 2018[5]).

Details about the homogeneous road segments, previously identified on the layout under investigation, are provided below:

- homogeneous segment 1 : Length L_1 = 160.00 m, Slope. p_1 = 2.60%
- homogeneous segment 2 : Length C_1= 226.66 m, Slope p_2 = 4.93%
- homogeneous segment 3 : Length L_{2a} = 160.00 m, Slope p_3 = 5.00%
- homogeneous segment 4 : Length L_{2b} = 160.00 m, Slope p_4 = 0.03%
- homogeneous segment 5 : Length C_2 = 169.16 m, Slope p_5 = 4.60%
- homogeneous segment 6 : Length L_3 = 163.41 m, Slope p_6 = 4.60%
- homogeneous segment 7: Length C_3 = 160.00 m, Slope p_7 = 1.20%
- homogeneous segment 8: Length L_{4a} = 160.73 m, Slope p_8 = 1.20%
- homogeneous segment 9: Length L_{4b} = 211.47 m, Slope p_9 = 3.32%
- homogeneous segment 10: Length C_4 = 160.00 m, Slope p_{10} = 3.32%
- homogeneous segment 11: Length L_5 = 210.31 m, Slope p_{11} = 3.32%

13.5 Diagnosis

As stated in the introduction, this site was selected because it resulted as having high potential for safety improvement from a simulated Network Screening stage (HSM manual, 2010 - Chapter 4[1]). Hence, this segment has been deeply investigated.

The second step of the safety analysis and management process is "Diagnosis", as described in the HSM manual, 2010 - Chapter 5[1]. The diagnosis goal is to identify crash causes and to learn lessons for possible improvements. The diagnosis process consists of:

- step 1 - safety data review;
- step 2 - assessment of supporting documentation;
- step 3 - assessment of boundary field conditions.

13.5.1 Safety data review

The site diagnosis starts from the analysis of past crash data. The crashes must be divided according to their severity, type, environmental conditions and location. The crash data analysis is useful to identify their causes as based on date, time, direction of travel, weather conditions and driver behaviours.

The data are generally collected in the crash report by law enforcement agencies. In this case, data in the period 2008-2014 collected by the local Police were obtained. Other data come from the Puglia Regional Agency for the Mobility (A.R.E.M.-now ASSET), agency which collects crash regional data in accordance with the National Institute of Statistics (ISTAT). In the study case, the, the location of the crashes is precise, the time window is short, but the information about the direction of travel is often absent. This last point is crucial for the crash reconstruction.

There are no indications about the drivers' fault in crashes because the information is synthetic and from different sources. However, data are only aimed at the engineering reconstruction for safety enhancement purposes.

The correct data analysis must consider:

- descriptive statistics of crash conditions;
- crash locations.

13.5.1.1 Descriptive statistics of crash conditions

The crash data are processed to be clustered and so to identify their potential causes/recurrent patterns. The descriptive statistics include:

- crash identifiers: date, day of week, time of the day, kilometer;
- crash Type, according to the HSM classification:
    - rear-end;
    - sideswipe;
    - angle;
    - turning;
    - head-on;
    - run-off the road;
    - fixed object - animal;
    - out of control;
    - work zone-related;
- crash severity, according to the KABCO scale defined by the HSM. However, based on crash reports three classes are considered: fata (K), injury (A/B/C), property damage only (O) crashes and the number of involved people;
- sequence of events: Direction of travel and the contributing circumstances;
- vehicles involved: one or more vehicles involved;
- pavement condition at the time of the crash: dry, wet, snow, icy;
- weather condition at the time of the crash: clear, cloudy, fog, rain, snow, ice.

An example of the review results is reported in the following table.

Tab. 13.3: Example of Crash report.

Crash	*Date*	*Day of the week*	*Time*	*Location*	*Severity*	*Type of crash*	*Road surface*	*Weather condition*	*Road geometry*	*Direction*	*Vehicle*	*Dynamics*	*Casualties*	*Injured*	*PDO*
9	/	Week day	Morning	km 7.00	B/C	out of control	Wet	Rainy	Tangent	-	Car	-	/	1	/
11	/	Week day	Night	km 7.70	B/C	out of control	Wet	Rainy	Curve	-	Car	-	/	1	/

Starting from the available number of crashes happened on this road segment during the fixed time window $n_{crashes}$, it is possible to estimate crash rates.

The observed crash rate is the ratio between the observed number of crashes over a fixed period (so the observed crash frequency) and the exposure (e.g., travelled km or the average traffic volume on a given segment for the same period used for the observed crash count). The equation is the following:

$$CR = \frac{n_{crashes}}{N_{observation\ period\ (years)} * AADT * 365 * Km} \qquad \text{(Eq. 13-5)}$$

The observed crash rate could be interpreted as the likelihood (based on past events) of being involved in a crash for each exposure unit of measurement. For example, the crash rate equal to 1 crash/1 million of vehicle per km, means that there is a 1/1’000’000 likelihood to be involved in a crash on 1 km of that road segment.

Only the fatal and injury crashes were considered because deemed the most reliable in the available data. There were 11 fatal and injury crashes occurred over 7 years: CR = $11*10^6/(7*4202*365*2) = 0.51$ fatal and injury crashes/(million vehicles*year*km).

This result, in terms of likelihood, means that there is a chance of being involved in fatal or injury crashes equal to 0.51 over 1 million.

13.5.1.2 Crash location

The analysis about the crash location could be run thanks to the collision or condition diagrams. These diagrams could highlight the location and the contributing circumstances.

Collision diagram

A collision diagram is a two-dimensional plan view (schematic) representation of the crashes that have occurred at a site within a given time period. It provides the crash type and crash location. The crashes are drawn by symbols which identify the vehicle type, the day, the time of the crash, the direction of travel, the road surface conditions as well as the weather conditions, type and severity.

The used symbols are suggested by the HSM manual (2010)[1], except for the symbol related to the “out of control” crash because almost all of the crashes occurred on the analysed road segment are due to “out of control” circumstances. Hence, using another symbol for this crash simplifies the graphic output. Another symbol for the “out of control” crash was reported at both the beginning of the “out of control” phenomenon (uncertain) and the end of it (detected by law enforcement agency).

The crash summary is listed on the diagram as well as in the report, including the total number of fatalities and injuries. The crash reconstruction is influenced by the lack of information about the direction of travel. An example of collision diagram is shown below.

Fig. 13.7: Example of Collision diagram- Curve 3 (Colonna et al., 2018[5]).

Condition diagram

The condition diagram is a plan view drawing of as many site characteristics as possible. The goal of this diagram is highlighting the road critical issues which could have contributed to the crash circumstances or caused the crash itself. For this reason, the condition diagram should be compared with the collision diagram. in the study case, the condition diagram focuses on the characteristic elements of the road segment which could have affected crash occurrence. These characteristic elements are the following:

- presence of dry-stone walls;
- maintenance status of the lateral ditch;
- absence of runoff management devices in the road cross section;
- road surface conditions (potholes, ruts, cracks, ponding);
- absence or inadequacy of the road barriers;
- presence of trees, vegetation or obstacles on the adjacent land area;
- presence of curve warning signs (Fig. II 468 C.d.S.[8]).

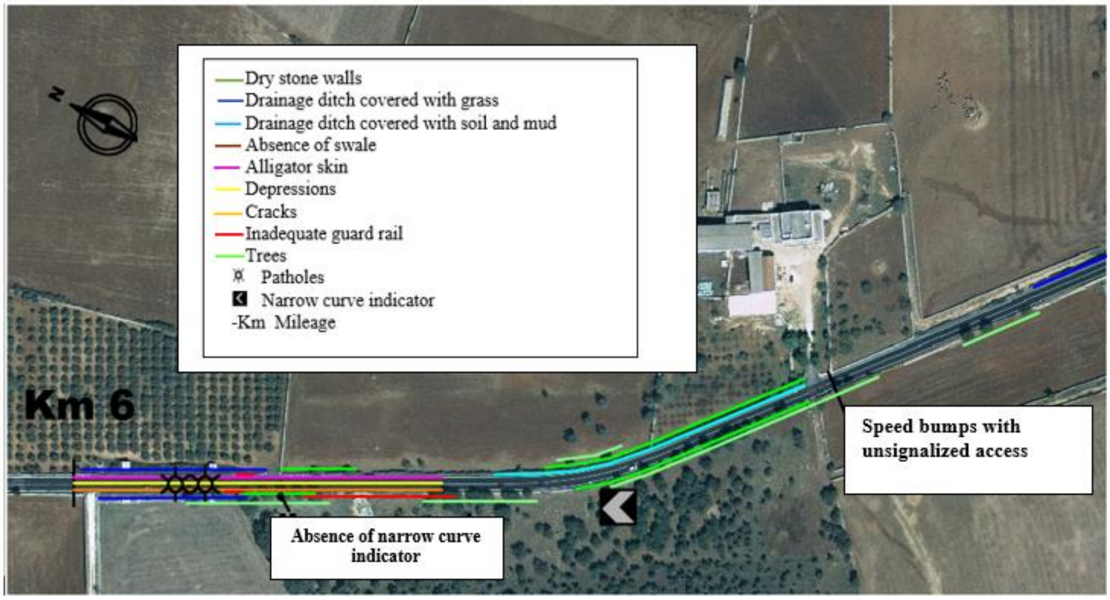

Fig. 13.8: Example of Condition diagram- Curve 1 (Colonna et al., 2018[5]).

[8] Legislative Decree n. 285, 30 aprile 1992, "*Codice della strada*", Italian road regulations.

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The figure shows problems related to the curve 1: there is a crest vertical curve which causes problems to the drainage of water, so that the water is forced to stay on the road pavement. This is due to the presence, in that point, of two accesses, whose slope is towards the main road, and so the runoff from the accesses goes directly to the main road.

13.5.1.3 Assess supporting documentation and traffic data

The assessment of the supporting documentation is the second step of the overall diagnosis of a site. The goal of this assessment is to obtain and review documents or experiences from local transportation professionals, stakeholders or maintainers that provide additional perspectives to the crash data review. In this phase, it could be possible to review past site documentation which provides some historical context about the crashes.

The following types of information may be useful as supporting documents to a site safety assessment (as stated by the HSM manual, 2010[1]):

- as-built construction plans;
- current traffic volumes for all directions of travel (and recent transportation studies, if available);
- relevant design criteria and guidelines;
- inventory of field conditions, also supported by photos and videos;
- maintenance logs;
- land use mapping and traffic access control characteristics (considering also existing improvement plans);
- weather patterns;
- other information.

In the study case, on the Provincial Road built decades ago, as-built construction plans were reproduced as aforementioned. The design criteria and the comparison with the current regulations (D.M. 2001[9] and D.M. 2006[10]) will be discussed after.

The traffic volume analysis has been run on detections made in 2007, other data being unavailable. The detections were made on both the direction of travels[11], counting 2’135 vehicles (including heavy vehicles). The detected operating speed, V_{85} is 127 Km/h. In the opposite direction, 2’067 vehicles were detected (including heavy vehicles) with operating speed, V_{85}, equal to 118 Km/h. The total number of travelling vehicles is 4’202 (including heavy vehicles).

13.5.2 On-site inspections (D.Lgs. 35/2011[12])

The diagnosis can be supported by a field investigation. Field observations can serve to validate safety concerns identified by a review of crash data or supporting documentation. The Appendix C to the HSM manual (2010) - Chapter 5[1] includes guidance on how to assess field conditions. However, this study relies on the “Linee guida per la gestione della sicurezza delle infrastrutture stradali” (“Guidelines for safety management of road infrastructures”)[13] on how to assess boundary field conditions, because this standard is closer to the Italian reality.

The Guidelines were issued in 2012, pursuant to the article 8 of the Decreto legislativo (Legislative decree) n. 35/2011, which implements the EU Directive 2008/96/CE on safety management of road infrastructures. These guidelines set the criteria and the modalities on how to: 1) make safety checks on design projects and 2) perform safety inspections of existing infrastructures, 3) rank roads according to their safety performances. Safety inspections of existing infrastructures, defined as “road safety analyses” are a preventive process. Their aim is to identify potential safety problems, both of new infrastructures and of adjustments on existing infrastructures, and the road operating characteristics. Safety inspections exist for urban and rural/suburban roadways and for

[9] Ministerial Decree n. 6792 of 5 November 2001, *Functional and Geometric Road Standards -Norme Funzionali e Geometriche delle Strade*.

[10] Ministerial Decree of 19 April 2006, *Functional and geometric standards for the construction of road intersections - Norme funzionali e geometriche per la costruzione delle intersezioni stradali*.

[11] Regione Puglia (2008), *Piano Regionale dei Trasporti*.

[12] Legislative Decree 15 March 2011 n. 35: *Actuation of the European directive 2008/96/CE about the road infrastructure management - Attuazione della direttiva 2008/96/CE sulla gestione della sicurezza delle infrastrutture stradali*.

[13] Ministerial Decree n. 137 of 2 May 2012, *Guidelines for the management of road infrastructure safety pursuant to art. 8 of Legislative Decree no. 35 of 15 March 2011 - Linee guida per la gestione della sicurezza delle infrastrutture stradali ai sensi dell’art. 8 del decreto legislativo 15 marzo 2011, n. 35*.

different road cross sections. The case study deals with a two-way two-lane rural roadway.

Safety inspections are “punctual” or detailed, for critical (or potentially critical) points and “diffused” for each homogenous road segments. As prescribed by the guidelines, all the inspections must be run for both the direction of travels and during day-time and night-time.

The diffused inspection goal is to obtain information about the user perception of the surrounding landscape conditions and specific details about road characteristics. The inspections are run along the main road and at singular points (like intersections, accesses, service roads, safety barriers and so forth). A specific inspection methodology and equipment are required for the punctual inspections, though.

The diffused inspection consists of a preliminary inspection and of a general one. The inspections were run on-board of a vehicle travelling at 50 Km/h, in order to catch all the useful details for both the directions, as stated by the guidelines.

The diffused inspection was taped by a camera and supported by pictures too, in order to enable the inspector to make further analyses.

13.5.2.1 Preliminary inspection

The main goal of the preliminary inspection is to analyse the interaction between the user and the surrounding landscape conditions. The inspection sheet is filled with the road identification (name, coordinates, homogenous segment length) and other general annotations. The inspection sheet filled for both the directions of travel is divided into: Macro-items, items, parameters and metrics according to a hierarchical table.

Macro-items and items are constant in the same environment (urban or rural), while other parameters vary with the road type. The macro-items are general aspects (critical boundary conditions, traffic, surrounding landscape conditions, speed and traffic sign system) and road geometry (horizontal and vertical alignments and the coordination between them). The inspector provides qualitative and descriptive evaluations of them.

The detailed analysed items are shown below.

- **Item:** “Critical boundary conditions”.
  - **Parameter** “*Road surface condition”:* two different road pavements were used on this road. From km 6.200 to km 7.200 a permeable asphalt pavement was implemented on the road layout. The other kilometres of the road are covered by a regular impervious asphalt pavement, even if it is often damaged by ruts, potholes and cracks.

Fig. 13.9: Example of Road surface conditions.

- **Item:** “Traffic”.
  - *Parameter* - Types of facility users: the recorded average daily traffic is 4’202 vehicles per day (2007). The 6% of this volume is a heavy-vehicle traffic.
- **Item:** “Surrounding landscape conditions”.

- *Parameter* - Roadside: its width is shallow and invaded constantly by trees and dry-stone walls (sometimes the trees are in the dry-stone walls).

Fig. 13.10: Example of trees and dry-stone walls.

- *Parameter* - *Buffer zone*: along the whole road layout there are trees preventing an optimal sight distance for drivers or hide the vertical traffic signs.

Fig. 13.11: Example of vertical traffic sign hidden by the trees.

- **Item:** "Speed".
  - *Parameter* - Design speed - posted speed: the posted speed limit is 60 Km/h; but the difference between the design speed, obtained by the speed diagram, and the posted speed limit is higher than 30 km/h in some segments and in other segments it is higher than 40 Km/h.
  - *Parameter* - Posted speed, Operating speed (V_{85}): the difference between the operating speed, calculated through appropriate models and the posted speed limit is often higher than 30 Km/h, but sometimes higher than 60 km/h indeed. The highest differences are detected on the tangents approaching to the analysed road segment: in fact, from the inspections, it was evident that the operating speed on the longest tangent is 118 Km/h, direction: Noci and 127 Km/h, direction: Gioia del Colle.
- **Item:** "Traffic sign system".
  - *Parameter* - Vertical traffic signs: there are no mileposts; the curve warning sign at the Curve 1 (direction: Noci) is absent; not all accesses are provided with appropriate vertical signs.
- **Item:** "Horizontal alignment".
  - *Parameter* - Tangent: there is a unique tangent 6 km long (from km 0 to km 6.300) interrupted by a signalized intersection at around 3.7 km from the Curve 1.

- *Parameter* - Transition curve: there is a short spiral transition curve at the Curve 3.
- *Parameter* - Circular curve: the radius of the Curve 4 (Km 7.700) is inadequate. The radius of following curves is inadequate as well.

- **Item:** "Vertical alignment".
  - *Parameter* - Crest vertical curve: the vertical curve is marked at the km 6.500, not respecting the minimum length. Moreover, there are two accesses on it, without appropriate road signs.
- **Item:** "Horizontal-vertical alignment coordination".
  - *Parameter* - Alignment perception: the only evident problem is related to a slight false turn of the roadsides at Curve 3 (direction: Noci).

A summary of the preliminary inspection sheet (as provided by the cited guidelines) is reported in the following table.

Tab. 13.4: Preliminary inspection sheet (Colonna et al., 2018[5]).

Macro-area	*Item*	*Parameter*	*Indicator*	*Judgement (to be filled by the road inspector)*
General Features	Critical Weather Conditions	Weather (Fog, Wind, Snow, Rain)	Lack or insufficient advices to users	✓
			Inadequate countermeasures	✓
	Traffic	Road Pavement Conditions (Ice, Water Flooding, Rubbles)	Lack or insufficient advices to users	✓
			Inadequate countermeasures	✓
		Volume	Inadequate cross-section	✓
		Type	Presence of specific components	✓
	Surrounding Environment	Clear Zones	Presence of obstacles, dangers, service, roads, etc.	✓
		Clear Zones (Out of the Fences)	Presence of buildings, trees, etc.	✓
		Beyond Clear Zones	Distraction for particular problems, other roads, etc.	✓
	Speed	Design Speed-Operating Speed	Excessive difference (+/-)	✓
		Maximum Posted Speed-Operating Speed	Excessive difference (+/-)	✓
	Road Signs	Horizontal Road Signs	Not homogeneous	✓
		Vertical Road Signs	Not homogeneous	✓
		Variable Message Signs	Ineffective information	✓
Geometry	Horizontal Alignment	Tangent	Excessive lengths	✓
		Transition Curves	Absence or inadequate transition curves	✓
		Circular Curves	Inadequate radius of curvature	✓
	Vertical Alignment	Slopes	Excessive slopes	✓
			Excessive lengths	✓
		Crest Vertical Curves	Presence of crest vertical curves	✓
		Sag Vertical Curves	Presences of sag vertical curves	✓
	Perception	Perception	Incorrect sight perception	✓
			Losing perception of road layout	✓

13.5.2.2 General inspection

The general inspection goal is to identify specific concerns along the alignment, according to the road type and the context where it is placed.

The specific aspects are then associated to a georeferenced location.

In the first part of the general inspection sheet, as well as in the preliminary inspection sheet, all data from inspectors are reported together with the road identification aspects.

The second part of them must be filled by checkmarks (after having checked on site the state-of-the-art of the parameters).

In the previous table, the checkmarks stand for a safety concern of growing severity (M= medium concern, the gap is filled by grey checkmarks, G = severe concern, the gap would be filled by dark grey checkmarks). This procedure is highly subjective, but the inspector tries to make an objective estimation of the road concerns for each observed parameter, according to what he/she has detected.

Tab. 13.5: Example of general inspection sheet (first part of the sheet, direction: Noci).

General information about the road		SP 239 (Ex SS 604) - secondary rural road ("C2")			
Milepost (Km)		Start: km 6		End: km 8	
	Day-time	*Night-time*	*Date*	*Starting Time*	*End Time*
1st Inspection	X		08/05/15	10:40	10:43
2nd Inspection		X	23/05/15	21:36	21:39

Macro-item	*Item*	*Parameter*	*Metric*	*km*	*0.2*	*0.4*	*0.6*	*0.8*	*1.0*
Rodway	Platform, Curbs and Buffer Zone	Side Shoulder	absence or insufficient width	M					
				S	✓	✓	✓	✓	✓
			narrowing close to attractive areas for pedestrians	M					
				S					
		Right Lane	insufficient width	M	✓	✓	✓	✓	✓
				S					
			Excessive width	M					
				S					
		Median	absence	M	✓				✓
				S					
			inadequacy of type	M	✓				
				S					
			inadequacy of transition or beginning part	M					
				S			✓		
			incorrect installation	M					
				S					
			unprotected obstacles	M					
				S					
		Scarps	insufficient maintenance of vegetation	M					
				S					
			absence of protective devices	M					
				S					
		Drainages	inefficient maintenance	M	✓		✓	✓	✓
				S		✓			
		Fence	inefficient maintenance	M					
				S					
Signage	Horizontal Signs	Edge Line Visibility	insufficient retro-reflectivity	M					
				S					
		Lane Lines Visibility	insufficient retro-reflectivity	M					
				S					
		Special Point Guide	Absence or inadequacy	M					
				S					
		Centreline	inadequacy for the steering manoeuvre	M					
				S					
	Vertical Signs	Danger Signs Prescription Signs Indication Signs	Insufficient visibility	M		✓	✓		
				S					
			inadequacy legibility	M					
				S					
			inadequate intelligibility	M					
				S					
		Speed Limits	absence or incorrect location	M					
				S					
			inadequacy according to the design speed	M	✓	✓	✓	✓	✓
				S					
			inadequacy with respect to the operating speed	M					✓
				S	✓	✓	✓	✓	
	Light Signals	Danger Signs Prescription Signs	inefficient maintenance	M					
				S					
		Right lane/tunnel Portals/flashing Lamps	inefficient maintenance	M					
				S					
		Margin Delineators	inefficient maintenance	M					
				S					
	Complementary Signals	Curve Delineators	absence or inadequacy	M					
				S					
		Edge Delineators	absence or inadequacy	M	✓	✓			
				S			✓	✓	✓

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The inspection sheet, in this part, is divided in grids for segments whose length is 200 m for each. The maximum covered length for each inspection sheet is 2 km (i.e., ten grids).

This sheet is divided in Macro-items, items, parameter and metrics as well. The macro-items are: *Road section, traffic signs, accesses, road pavements, lighting systems, other aspects*.

The following aspects are remarkable (severe concerns):

- the shoulder width is insufficient;
- high differences between posted speed limits, design speeds and operating speeds;
- lane markers are absent or illegible, as highlighted during night-time inspections;
- absence of rest areas (they should be at a maximum distance of 1 km, according to D.M. 5/11/2011, n.6792[9].

Moreover, there is the already mentioned problem of the two accesses, without road signs, at the crest vertical curve (km 6.500), which cause water run-off problems. Run-off water is full of debris and mud which make the road pavement slippery and affect drainage, accelerating the clogging effect of the permeable pavement and impeding the flow in the lateral ditches. The general inspection sheet is made of two sheets for each direction of travel. The starting point of these sheets is the km 6.000.

13.5.2.3 Punctual inspection

Starting from the critical points expressed by the previous inspections is possible to deepen the analysis through the punctual inspections. They must be run during both nigh-time and daytime, to identify the safety concerns at some specific points, where a high number of crashes has occurred (related to the exposure) or where there are critical issues highlighted by the diffused inspections. The punctual inspections are run at all the singular points of the road layout, using the ad-hoc inspection sheets: intersections, important interferences (tunnels, bridges, etc.).

In this case, only already highlighted critical points have been analysed because of a lack of singular points.

13.5.3 Geometric checks

In this paragraph, a critical analysis of the road layout is provided according to the regulations, particularly the Italian road standards D.M. 05/11/2001[9]. The examined road was designed before the introduction of the current regulation and this can likely be the reason why some criteria from the regulation could be not met.

Fig. 13.12: General view of the context where the road segment (in red) is placed (photo source Open Street Map).

The examined road is categorized as a secondary rural road, “C2” with a design speed range between 60 Km/h and 100 Km/h. Albeit the road is geometrically close to a local rural road, “F2”, its function in the general context is not comparable with an “access road”, like local roads. The main function of this SP 239 is to carry the traffic volume towards the local road network, after having analysed the whole network. In fact, vehicular flows coming from the Autostrada A 14 (Freeway) and the Strada Statale 100 (State roadway), near Gioia del Colle, must converge on the SP 239 to reach one of the destinations in the Valle d’Itria area. The same consideration is valid for the flows coming from the Murgian inland (Altamura, Santeramo in Colle, etc.) heading to the Valle d’Itria area. The analysed direction of travel is Gioia del Colle - Noci.

13.5.3.1 Checks on the horizontal alignment

The geometric characteristics of a secondary rural road "C2" (labelled as "DM") are shown and compared with the geometric characteristics of the SP 239 as follows:

Lane: $1 = 3.00$ m ÷ $1_{DM} = 3.50$ m -> *NOT VERIFIED*
Shoulder: $1 = 0.50$ ÷ $1_{DM} = 1.25$ -> *NOT VERIFIED*
Total carriageway Width = 7.00 m
$Vp_{,min} = 60$ Km/h
$Vp_{,\,max} = 100$ Km/h
$i_{c,\,min} = 2.50\%$ (cross slope on road tangents) -> *VERIFIED*
$i_{c,\,max} = 7.00\%$ (cross slope on curves) -> *VERIFIED*

Tangent checks

The tangent length is included between a minimum and a maximum value:

$$L_{min} < L < L_{max} \qquad \text{(Eq. 13-6)}$$

where:

$$L_{max} = 22 \; x \; Vp_{,max} \qquad \text{(Eq. 13-7)}$$

so equal to $L_{max} = 22 \; x \; 100 \frac{km}{h} = 2200 \; m$

Instead L_{min} depends on the maximum speed taken from the speed profile. The following table shows that the maximum tangent length is not satisfied for the "tangent 1".

Tab. 13.6: Tangent checks.

Tangent	*V_{max} (Km/h)*	*L (m)*	*L_{min} (m)*	*Check: L_{min}*	*Check: L_{max}*
Tangent 1	100	6000.00	150.00	Verified	*Not Verified*
Tangent 2	100	356.96	150.00	Verified	*Verified*
Tangent 3	94	197.21	118.50	Verified	*Verified*
Tangent 4	91	403.11	129.00	Verified	*Verified*
Tangent 5	100	1760.00	150.00	Verified	*Verified*

The cross slope of the road segment is 2.50% on the tangents, so it is always verified.

Circular curve checks

Tangent and curve sequence

A tangent long Lr between two curves, whose minimum radius is R, must be consistent with the following standardized relations:

$$\text{If } L_R < 300m \rightarrow R > L_R \qquad \text{(Eq. 13-8)}$$

$$\text{If } L_R > 300m \rightarrow R \geq 400m \qquad \text{(Eq. 13-9)}$$

The checks of these requirements show that there is only one case in which the relation is not verified.

Tab. 13.7: Tangent-curves consistency.

Tangent	*$L_{tangent}$ (m)*	*R_{Prec} (m)*	*R_{Succ} (m)*	*R_{DM}*	*Check*
Tangent 2	356.96	$R_1 = 170.00$	$R_2 = 290.00$	$R_1 > 400m$	*Not Verified*
Tangent 3	197.21	$R_2 = 290.00$	$R_3 = 160.00$	$R_3 > L_{r3}$	*Not Verified*
Tangent 4	403.11	$R_3 = 160.00$	$R_4 = 80.00$	$R_4 > 400m$	*Not Verified*

Length of curves

A circular curve must be correctly perceived by the driver and so it should have a length to be travelled in at least 2.5 seconds, in relation to the design speed of the curve. The following equation must be respected, indeed:

$$L \geq \frac{V_p}{3.6}\, x\, t \qquad \text{(Eq. 13-10)}$$

In the case study, this relation is not respected twice out of four times, as reported in the following table.

Tab. 13.8: Checks about the minimum length of circular curves.

Curve	*Length (m)*	*Minimum length (m)*	*Check*
Curve 1	23.56	48.05	*Not Verified*
Curve 2	57.09	58.88	*Not Verified*
Curve 3	94.95	46.94	*Verified*
Curve 4	35.68	35.53	*Verified*

Radius of circular curves

The general equation about the design speed linked to the curve radius, the friction coefficient and the cross slope of the curve is the following:

$$\frac{V_p^2}{R\, x\, 127} = q + f_t \qquad \text{(Eq. 13-11)}$$

The minimum value of the radius (R) is computed according to the previous equation. With the design speed (Vp) equal to 60 Km/h, friction coefficient (f_t) equal to 0.17 (value influenced by the speed), the minimum radius is 118 m. In the case study, the radius is less than the minimum value (118 m) in one out of four cases. All the cross slopes are less than 7% (Curve 2 has a cross slope less than 6%).

Tab. 13.9: Checks about radius and cross slopes.

Curve	*Radius (m)*		*Check*	*q_t (%)*	*Check*
Curve 1	170	$R_{min} < R < R^*$	*Verified*	6.73	*Not Verified*
Curve 2	290	$R_{min} < R < R^*$	*Verified*	5.60	*Not Verified*
Curve 3	160	$R_{min} < R < R^*$	*Verified*	6.20	*Not Verified*
Curve 4	80	$R < R_{min}$	*Not Verified*	6.46	*Not Verified*

In the following graph taken from the D.M. 6792/2001[9], the characteristic radius values are shown, together with the radii values of this case study (R1, R2, R3, R4).

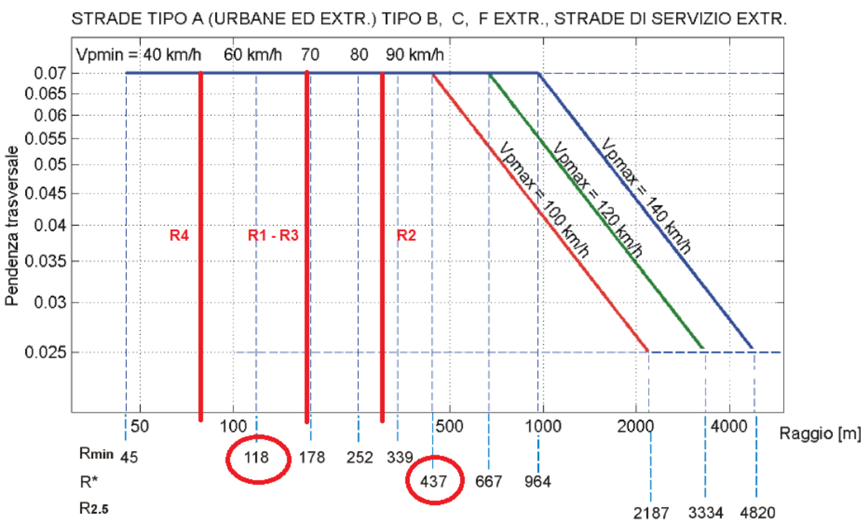

Fig. 13.13: Abacus n. 5.2.4.a, taken from the D.M. 11/05/2001[9], having the radius of curvature on the x-axis (m), the cross slope on the y-axis (-) for different values of design speeds (Vpmin = minimum design speed, Vpmax = maximum design speed). The maximum design speed for this case study is 100 km/h (red diagonal line in the figure).

Consistency of subsequent curve radii

Two following curves must have radii whose ratio is in accordance with the standardized abacus (D.M. 6792/2001[9]). This abacus suggests for each road category what is the accepted range for the ratio between subsequent radii or the optimal range (just for "A" and "B" primary roads).

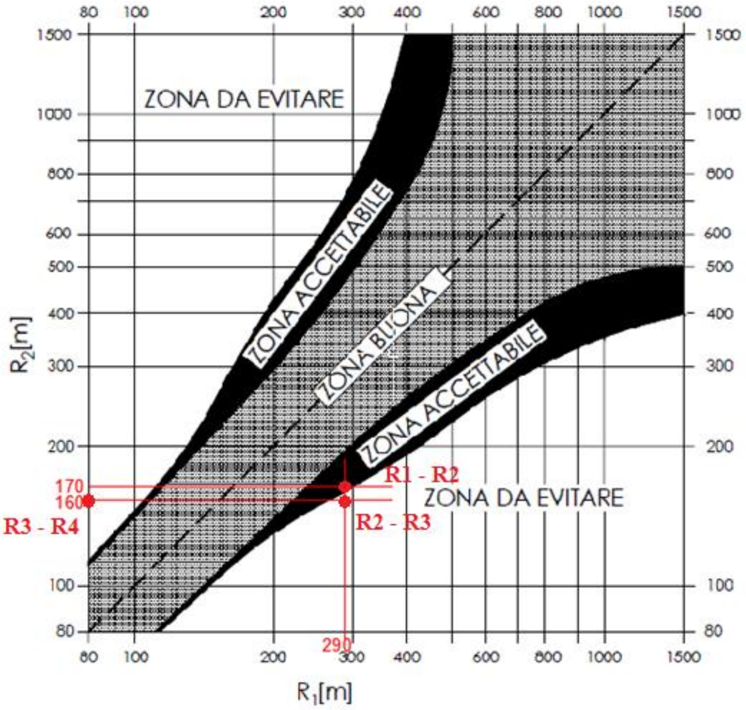

Fig. 13.14: Abacus n.5.2.2.a, D.M. 11/05/2001[9] ("Zona da evitare" = unacceptable ratio between two subsequent curves, "zona buona" = good ratio, "zona accettabile" = acceptable ratio).

The following table shows that two out of three checks are not verified.

Tab. 13.10: Checks about the curve radius consistency.

Elements	*Radius (m)*	*Diagram area*	*Check*
R_1 - R_2	170 - 290	Accepted	*Verified*
R_2 - R_3	290 - 160	To be avoided	*Not Verified*
R_3 - R_4	160 - 80	To be avoided	*Not Verified*

Transition curves checks

Criterion number 1 (Reducing the effect of a sudden lateral acceleration due to the circular curve)

Along the transition curve, the lateral acceleration must vary gradually to avoid the "kickback" effect. So, the parameter A of the transition curve must be greater than a minimum value related to the maximum speed V (km/h), obtained from the speed profile, according to the following equation:

$$A \geq A_{min} = \sqrt{\frac{V^3}{c} - \frac{g\,V\,R\,(q_f - q_t)}{c}} \qquad \text{(Eq. 13-12)}$$

Neglecting the second addendum of the equation, and assuming the kickback limit value:

$$c_{max} = \frac{50.4}{V}\left(\frac{m}{s^3}\right) \qquad \text{(Eq. 13-13)}$$

The parameter A is:

$$A \geq 0.021\, x\, V^2 \qquad \text{(Eq. 13-14)}$$

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Tab. 13.11: Checks about the parameter A - Kickback.

Spiral curve	*A*	*V (Km/h)*	*A_{min}*	*Checks*
Spiral curve 1-1	80.00	73.804	114.40	*Not Verified*
Spiral curve 1-2	90.00	75.030	118.20	*Not Verified*
Spiral curve 2-1	100.00	88.659	165.10	*Not Verified*
Spiral curve 2-2	150.00	93.774	184.70	*Not Verified*
Spiral curve 3-1	50.00	69.601	101.70	*Not Verified*
Spiral curve 3-2	50.00	69.644	101.90	*Not Verified*
Spiral curve 4-1	50.00	55.732	65.20	*Not Verified*
Spiral curve 4-2	50.00	55.730	65.20	*Not Verified*

The eight spiral curves of the case study are not compliant with the minimum value of the parameter A, for the kickback effect reduction.

Criterion number 2 (gradual tangent-to-curve cross slope variation)

At the starting and ending sections of the spiral curve, the road has different cross slopes that must be longitudinally connected, introducing the super-elevation of the curb.

If the spiral curve starts from a tangent (infinite radius) the parameter A must be compliant with:

$$A \geq A_{min} = \sqrt{\frac{R}{\Delta i_{max}} \; x \; 100 \; x \; B_i \; (q_i + q_f)} \qquad \text{(Eq. 13-15)}$$

Where:

Bi = distance between the rotation line and the road curb at the starting section of the transitional spiral curve.

Δ_{imax} (%) = 18* (Bi/V) = maximum longitudinal super-elevation of the points which are distant Bi from the rotational axis (V = design speed in Km/h).

In the following table, all the checks about the second criterion performed for the 8 transitional spiral curves are summarized: four spiral curves are not compliant with the minimum A parameter value.

Tab. 13.12: Parameter A - longitudinal super-elevation.

Spiral curve	**A**	**A_{min}**	**Checks**
Spiral curve 1-1	80.00	80.20	*Not Verified*
Spiral curve 1-2	90.00	80.90	*Verified*
Spiral curve 2-1	100.00	107.6	*Not Verified*
Spiral curve 2-2	150.00	110.6	*Verified*
Spiral curve 3-1	50.00	73.40	*Not Verified*
Spiral curve 3-2	50.00	73.40	*Not Verified*
Spiral curve 4-1	50.00	48.50	*Verified*
Spiral curve 4-2	50.00	48.50	*Verified*

Criterion number 3 (Optical)

In order to ensure the correct optical perception of the transitional spiral curve, the following relationship must be verified:

$$A \geq R/3 \qquad \text{(Eq. 13-16)}$$

Moreover, for the correct perception of the following circular curve, the following requirement must be verified too:

$$A \leq R \qquad \text{(Eq. 13-17)}$$

The table summarizing these checks about spiral curves is shown below.

Tab. 13.13: Checks about the parameter A - optical criterion.

Spiral curve	*A*	*R/3 (m)*	*R (m)*		*Checks*
Spiral curve 1-1	80.00	56.67	170.00	R/3 < A < R	*Verified*
Spiral curve 1-2	90.00	56.67	170.00	R/3 < A < R	*Verified*
Spiral curve 2-1	100.00	96.67	290.00	R/3 < A < R	*Verified*
Spiral curve 2-2	150.00	96.67	290.00	R/3 < A < R	*Verified*
Spiral curve 3-1	50.00	53.33	160.00	A < R/3	*Verified*
Spiral curve 3-2	50.00	53.33	160.00	A < R/3	*Verified*
Spiral curve 4-1	50.00	26.67	80.00	R/3 < A < R	*Verified*
Spiral curve 4-2	50.00	26.67	80.00	R/3 < A < R	*Verified*

Cross section enlargement at curves

In order to ensure the vehicle inscription in curves, with the needed interspaces between the vehicle and the lane edge lines, the circular curves must have each lane enlarged of a value E determined by the following:

$$E = \frac{K}{R} \quad [m] \qquad \text{(Eq. 13-18)}$$

Where K is 45 and R is the external radius of the lane, in meters.

If E is less than 20 cm, the lane, in curve, has the same width assumed on the tangent. The case study highlighted the following:

- Curve 1, 2 and 3 are not provided with enlargements;
- Curve 4 shows an enlargement E equal to 3.30 m measured as the difference between the standard width of 7 m and the inner lane curb drawn.

Tab. 13.14: Checks about the curve enlargements, E.

Curve	*R_e (m)*	*E_{DM} (m)*	*E_{Site} (m)*	*Checks*
Curve 1	173.50	0.26	0	*Not Verified*
Curve 2	293.50	0.15	0	*Verified*
Curve 3	163.50	0.28	0	*Not Verified*
Curve 4	83.50	0.54	3.30	*Verified*

Analysing the transition spiral curve, the enlargement starts 7.50 m before the spiral curve, and it ends 7.50 m after the final section of the spiral curve. The whole length of the enlarged segment is Lz:

$$L_z = 2 \times 7{,}50 + L \qquad \text{(Eq. 13-19)}$$

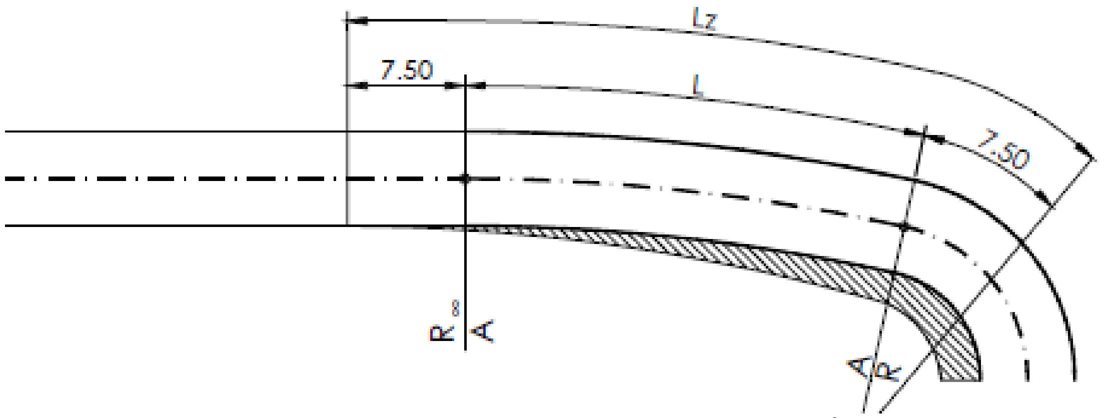

Fig. 13.15: Enlargement at the spiral curve (D.M. 6792/2001[9]).

In case of the Curve 4, the transition spiral curve 4.1 (on the left in the figure 13.16), ends 3.35 m before the spiral curve end then, it is not adequate. Instead, the transition curve 4.2 (on the right in the figure 13.16) has an enlargement which ends 11.14 m after the spiral curve end. Hence, it is adequate.

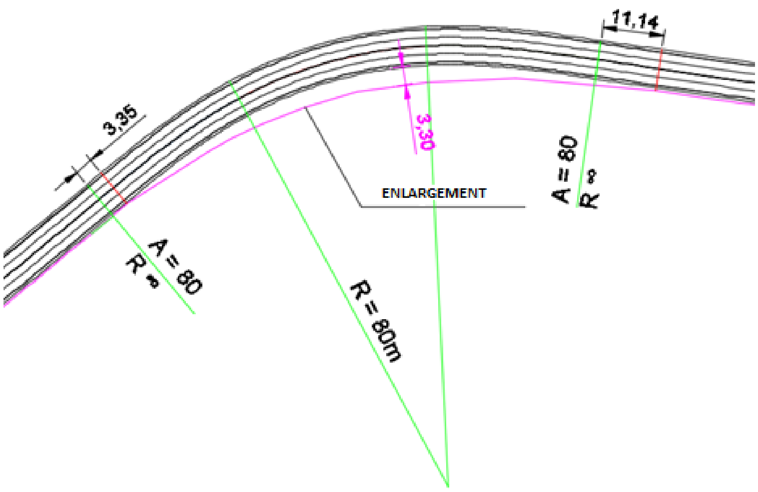

Fig. 13.16: "Curve 4" enlargement.

Rest areas checks

Rest areas are completely absent for the whole analysed segment. The current regulations, instead, provide the presence of rest areas at least every kilometre, on both the directions of travel.

13.5.3.2 Sight distance checks

For what concerns stopping sight distance, the investigated road segment shows several problems, as highlighted by red circles in the figures 13.17 and 13.18 for both directions of travel.

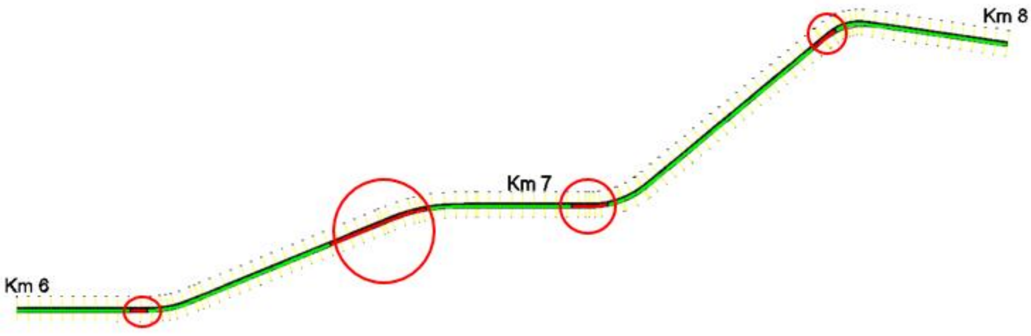

Fig. 13.17: Checks about the stopping sight distance (direction: Noci).

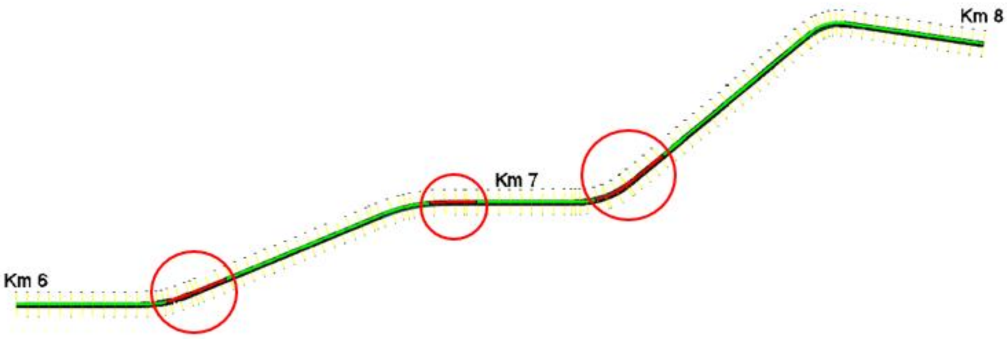

Fig. 13.18: Checks about the stopping sight distance (direction: Gioia del Colle).

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The overtaking manoeuvre is prevented by the straight edge centreline. This manoeuvre is allowed in the adjacent tangents and in other parts of the road layout, satisfying the minimum percentage set by the standard D.M. 6792/2001[9] equal to 20% of the entire route. The sight distance for the overtaking has been verified for this segment.

13.5.3.3 Speed profile checks

The posted speed on this segment is 60 Km/h but the recorded operating speed is greater than 100 Km/h. Since the speed is one of the main factors related to the crash severity, the speed profile obtained according to the Italian regulations D.M. 6792/2001[9] was reconstructed as based on the road geometry, instead of posted speeds. Doing so, the analysis is closer to the recorded operating speed, V_{85}. For the sake of safety, it is useless not considering the actual driver behaviours and analysing just the posted speeds.

- The analysis of the speed profile has led to highlighting the following issues: the transition length is always verified and less than the distance needed for recognizing the obstacles for the entire layout;
- there is an excessive difference between speeds of two subsequent geometry elements in the following cases:
  - 17.19 km/h is the greatest speed difference between two following curves, curve 2 and curve 3;
  - 48.83 km/h is the greatest speed difference between a segment (segment 5) and a curve (curve 4).

Tab. 13.15: Checks about the speed diagram.

ΔV checks according to maximum design speed Vpmax = 100 km/h					
Horizontal alignment road geometry	*Speed (km/h)*	*|ΔV| (threshold 10 km/h)*		*|ΔV| between curves (threshold 20 km/h)*	
Segment 1	100.00				
		30.81	NV		
Curve 1	69.19				
		30.81	NV		
ent 2	100.00			15.60	A
		15.21	NV		
Curve 2	84.79				
		9.50	V		
Segment 3	94.29			17.19	A
		26.69	NV		
Curve 3	67.60				
		23.28	NV		
Segment 4	90.88				
		39.71	NV	16.43	A
Curve 4	51.17				
		48.83	NV		
Segment 5	100.00				

Note - V: verified output; NV: not-verified output; A: Acceptable output

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13.5.3.4 Checks on the vertical alignment

From the vertical alignment analysis, some remarks are as follows:

- grades are always less than the maximum slope accepted by the standards, i.e. 7%;
- the grade number 4 has a longitudinal slope slightly marked, which was found to be 0.03%. The cross section is an embankment having the cross slope equal to 2.5%, with water flow ensured by the cross slope;
- the vertical curves and the sight distance related to them are often inadequate, as further shown;

Crest vertical curve checks

The following tables show the checks made on the crest vertical curves according to the Italian standards D.M. 6792/2001[9]. In the tables, the following variables are shown:

- length, in metres, of the vertical curve (L);
- the mean slope in percentage (i_{mean});
- the maximum speed in km/h (V_{max});
- the sight distance, in metres (D);
- the slope gradient, in percentage (Δi);
- the comparison between the sight distance and the length of the curve, D > L or D < L;
- the radius of the curve detected on site, in metres (R_{site});
- the minimum radius accepted according to the D.M. 6792/2001[9], in metres (R_{min});
- the check, if it is verified or not that R_{site} is greater than $R_{min.}$

Tab. 13.16: Crest vertical curve 1.

L(m)	*i_{mean} (%)*	*Vmax (km/h)*	*D(m)*	*Δi*	*D >/< L*	*Rsite (m)*	*Rmin (m)*	*Check*
97,04	-0.035	88.79	132.00	9.93	D > L	997.20	4630	*Not Verified*

Tab. 13.17: Crest vertical curve 2.

L(m)	*i_{mean} (%)*	*Vmax (km/h)*	*D(m)*	*Δi*	*D >/< L*	*Rsite (m)*	*Rmin (m)*	*Check*
94.28	-2.32	93.90	152.00	4.57	D > L	2064	5950	*Not Verified*

Tab. 13.18: Crest vertical curve 3.

L(m)	*i_{mean} (%)*	*Vmax (km/h)*	*D(m)*	*Δi*	*D >/< L*	*Rsite (m)*	*Rmin (m)*	*Check*
16.44	-2.86	8775	137.00	0.93	D > L	1777.49	All values	*Verified*

Sag vertical curve checks

The following tables show the checks made on the sag vertical curves, according to the Italian standards D.M. 6792/2001[9]. The same variables explained in the previous section, about the crest vertical curves, are expressed in the following tables.

Tab. 13.19: Sag vertical curve 1.

L(m)	*i_{mean} (%)*	*Vmax (km/h)*	*D(m)*	*Δi*	*D >/< L*	*Rsite (m)*	*Rmin (m)*	*Check*
59.03	1.165	90.95	134.50	7.53	D > L	783.89	3150	*Not Verified*

Tab. 13.20: Sag vertical curve 2.

L(m)	i_{mean} (%)	Vmax (km/h)	D(m)	Δi	D >/< L	Rsite (m)	Rmin (m)	Check
61.92	-2.515	98.10	167.00	4.97	D > L	1246.40	4070	*Not Verified*

Tab. 13.21: Sag vertical curve 3.

L(m)	i_{mean} (%)	Vmax (km/h)	D(m)	Δi	D >/< L	Rsite (m)	Rmin (m)	Check
54.95	-1.7	78.96	110.80	5.80	D > L	947.50	2500	*Not Verified*

Tab. 13.22: Sag vertical curve 4.

L(m)	i_{mean} (%)	Vmax (km/h)	D(m)	Δi	D >/< L	Rsite (m)	Rmin (m)	Check
97.77	-2.26	84.41	130.00	2.12	D > L	4606	100	*Verified*

13.5.3.5 Consistency of vertical and horizontal alignments

The consistency of vertical and horizontal alignments does not show particular critical situations, except for the curve 3 in the direction to Noci. This curve is immediately after the sag vertical curve, providing a false perception of the layout by the driver. This subsequence makes the driver perceive a greater radius than the actual one. This critical situation is demonstrated by the ratio between the vertical curve radius, R_v, and the horizontal curve one, R: 947.50m/160.00m = 5.96 < 6 (Not verified).

Fig. 13.19: Curve 3 (direction: Noci).

13.5.3.6 Checks of accesses

The D.M. 19/04/2006[10] defines an access as a "vehicle entry" from an area or a private building to the public road or vice-versa. The access location depends on the road category according to the "Codice della Strada" (Italian Road Regulations)[8].

The analysed road is a "C" road, so the regulation states a distance of at least 300 m between the accesses. Although this prescription, the road has been built before the D.M.[10], so the accepted inter-distance between two consequent accesses could be decreased down to 100 m, in specific cases. This value is the minimum allowed distance used by the road agency. The following figure/table shows the accesses location and inter-distances.

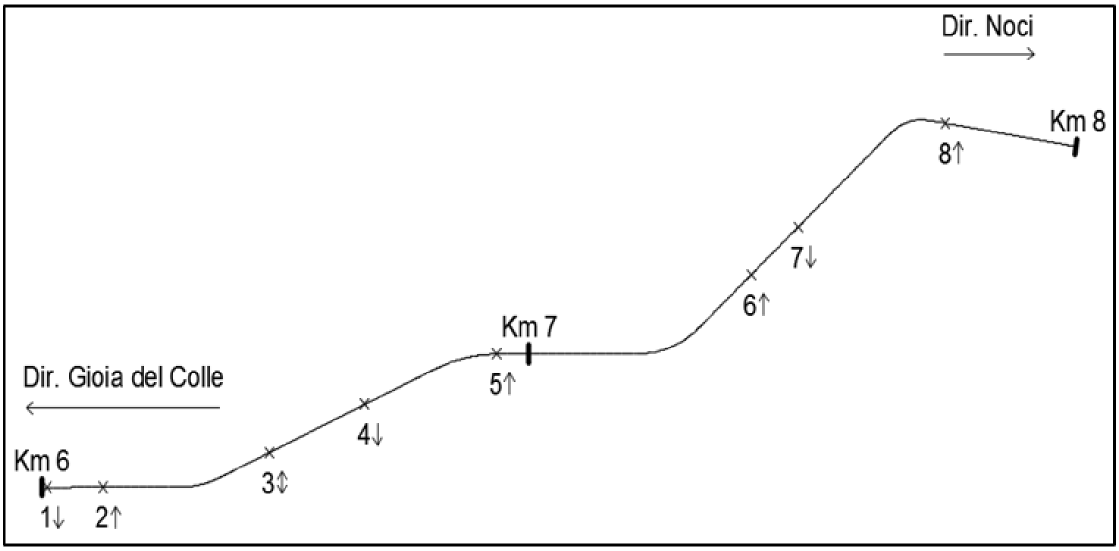

Fig. 13.20: Accesses locations.

Tab. 13.23: Access inter-distance checks.

Access	*Distance (m)*	*Check*
D_{1-2}	96.00	*Not Verified*
D_{2-3}	292.50	*Verified*
D_{3-4}	178.00	*Verified*
D_{4-5}	237.50	*Verified*
D_{5-6}	473.00	*Verified*
D_{6-7}	105.00	*Verified*
D_{7-8}	305.00	*Verified*

The distance between the first and the second access is not verified.

Stopping sight distance checks were repeated for each access, with respect to the conflict point indicated in Figure 13.21. The conflict point is defined by the intersection between the two centrelines of the two roads approaching the intersections. The other point that was considered is a point belonging to the road lane from which it is possible to see the obstacle drawing a tangent to the obstacle itself.

However, the actual conflict point has a variable position, due to the variability of the steering manoeuvre by the driver. However, the considered approximation is reliable because the output is precautionary among other possibilities. The following figure provides a graphic example of the sight distance.

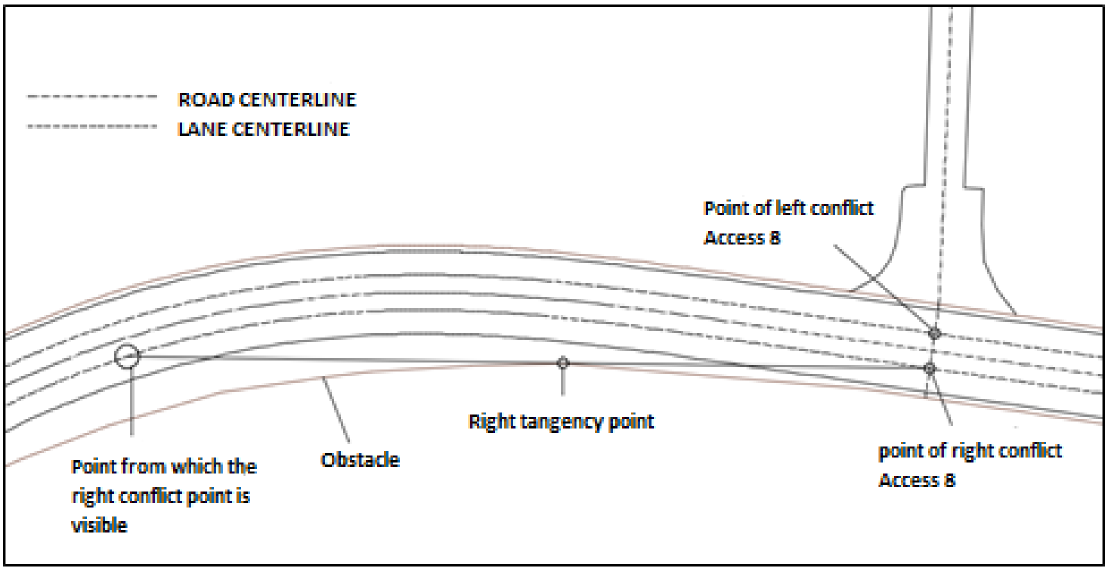

Fig. 13.21: Sight distance for the accesses.

The vertical sight distance was measured from the conflict point (which overlaps with the centreline of the access in the elevation profile), having height h_2, to the generic observer, having height h_1, drawing the tangent to the vertical curve. In the checks, the following heights were considered:

- height of the generic observer on the main road, $h_1 = 1.10$ m;
- height of the obstacle in the conflict point, $h_2 = 0.10$ m.

The assumed h_2 value is precautionary among different possibilities because the vehicle may have greater heights while steering than the one assumed for the check.

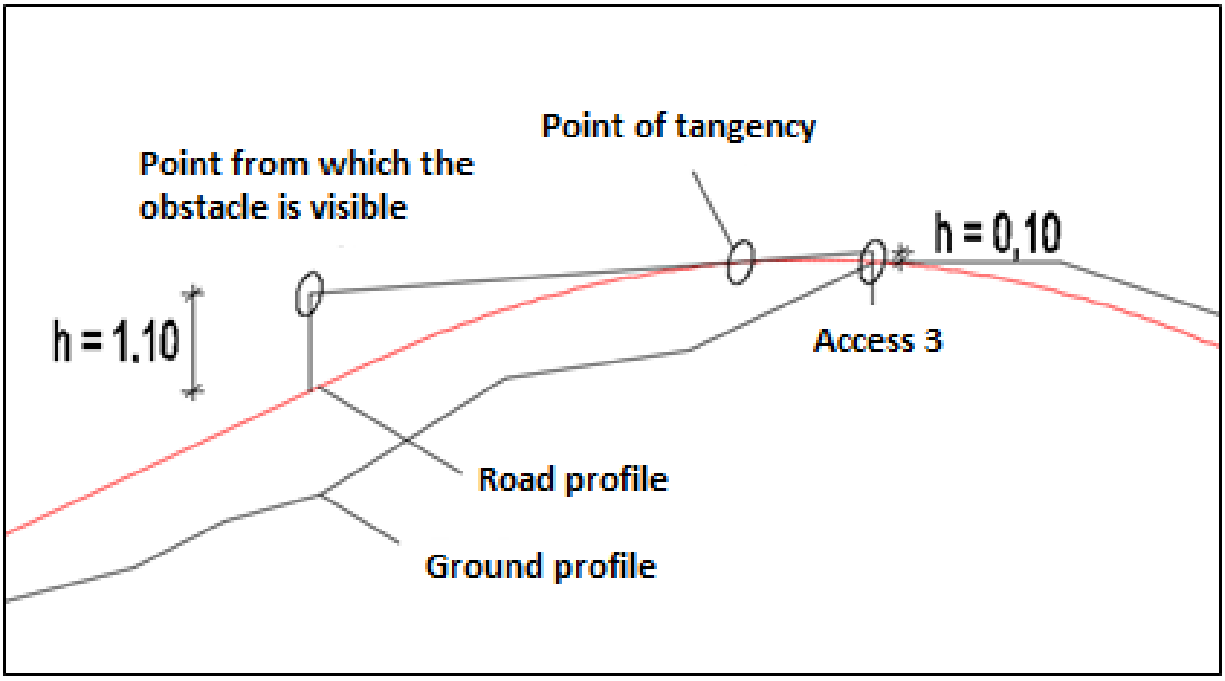

Fig. 13.22: Vertical sight distance for accesses.

The results are displayed in the table 13.24.

Tab. 13.24: Sight distance checks for accesses.

		Horizontal check				*Vertical check*			
Access	*Conflict*	*Milepost from which it is visible (m)*	*Sight distance (m)*	*Braking distance (m)*	*Result*	*Milepost from which it is visible (m)*	*Sight distance (m)*	*Braking distance (m)*	*Result*
1	left	269.48	277.98	87.44	✓	413.09	403.57	123.15	✓
1	right	--	600.00	164.00	✓	--	142.02	164.00	✗
2	left	271.30	201.37	87.44	✓	394.83	289.46	117.79	✓
2	right	--	600.00	164.00	✓	--	203.98	164.00	✓
3	left	764.86	383.05	121.99	✓	465.76	60.10	117.79	✓
3	right	252.83	149.89	88.94	✓	338.95	60.78	164.00	✓
4	left	772.05	205.00	121.99	✓	710.71	134.04	140.06	✗
4	right	260.47	309.19	87.44	✓	385.33	191.50	101.97	✗
5	left	1090..97	290.88	84.31	✓	--	1106.72	125.00	✓
5	right	736.55	77.45	121.99	✗	644.36	150.85	115.06	✓
6	left	1627.34	348.95	65.94	✓	--	635.01	164.00	✓
6	right	1123.01	183.98	84.31	✓	--	691.00	138.42	✓
7	left	1628.50	282.38	65.94	✓	--	531.02	164.00	✓
7	right	1126.51	297.85	84.31	✓	--	1028.79	164.00	✓
8	left	--	600.00	164.00	✓	--	225.81	164.00	✓
8	right	1627.98	67.92	65.94	✓	--	1357.08	164.00	✓

Left conflict: referred to the lane with direction from Noci to Gioia del Colle
Right conflict: referred to the lane with direction from Gioia del Colle to Noci

The accesses which do not comply at least with one horizontal or vertical sight distance requirement are:

- access 1, immediately after a crest vertical curve (from Gioia del Colle to Noci);
- access 3, there are two accesses, one in front of the other on the top of the crest vertical curve, whose radius was not appropriate, as emerged from checks for vertical curves (for both directions);
- access 5, it is in the middle of the spiral curve, after the horizontal curve (from Gioia del Colle to Noci).

Crash data cannot be linked to the access presence, though. Moreover, the braking distances have been calculated using the reconstructed design speed instead of the posted speed (60 Km/h). Hence, the following paragraphs will not deal with the countermeasures for the accesses (such as introducing frontage roads) but they will deal with the most severe crashes and the countermeasures to mitigate them. By the way, speeding issues will be considered, attempting to find countermeasures to contain speeds within the posted limits. Note that, to be accurate, sight distance triangles should also be drawn for accesses, as indicated by the Italian standards for intersections (D.M. 19/04/2006[10]).

13.5.4 Friction Diagram Method (FDM) along the road layout

The friction enables to transfer each force acting on the vehicle to the road pavement. A vehicle in motion has a "Friction Capital". If this value is exceeded, pure rolling occurs, and the vehicle is no more under control. The Friction capital, as already stated in Chapter 9 is the Friction Potential.

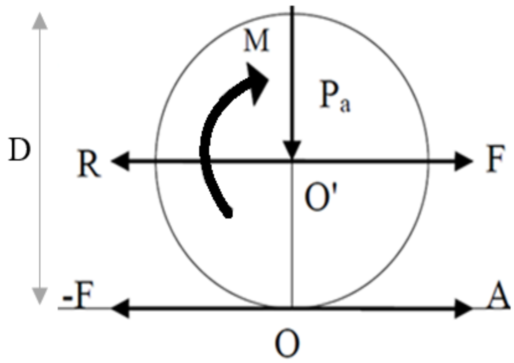

Fig. 13.23: Driving wheel.

The driving vehicle uses a part of the Friction Potential, the so-called Friction Demand according to the road geometry, tire wear, weather conditions, driving behaviours. Hence, according to changes in the boundary conditions, the Friction Demand varies too.

The ratio between the two quantities defines the Friction Used: a percentage of the Friction Demand over the Friction Potential. In this way, it is possible to know the safety margin towards skidding:

$$F_{USED} = \frac{F_D}{F_P} \, x \, 100 \, [\%] \qquad \text{(Eq. 13-20)}$$

Safety conditions are defined by the distance of the F_{USED} from its limit value which is 100%. Hence, in very safe situations, F_{USED} will have a low value; in situations in which the skidding risk is high, F_{USED} will have a value near 100%. While in situations in which vehicle stability is not ensured, F_{USED} will have a value greater than its limit.

In order to define the Friction Used, the calculation of all the actions transferred by the vehicle to the road pavement is needed, which depends on:

- road characteristics;
  - vertical-horizontal alignment;
  - road surface condition (dry, wet, dirty, etc.);
  - horizontal curve radii (circular or spiral curves);
  - vertical curve radii (sag, crest);
  - longitudinal slopes (uphill or downhill roads);
  - cross slopes.
- vehicle characteristics:
  - geometry and mass;

- tire resistances (rolling specific resistance).
- motion characteristics:
  - speed and acceleration in the considered section.

According to the combination of the aforementioned variables and other boundary conditions, some critical vehicles can be defined, with respect to reaching the Friction Used equal to 100%. The following table shows the critical vehicles derived from the motion characteristics according to the speed profile and the horizontal-vertical alignment (see Chapter 9).

Tab. 13.25: "Design critical vehicle" for each possible combination.

Combination	*Constant Speed*	*Acceleration*	*Braking*	*Wheel Drive*
Flat terrain tangent	I	I	II	Front
Uphill tangent	III	III	IV	Front
Downhill tangent	III	I	II	Rear
Tangent in crest vertical curve	III	III	IV	Front
Tangent in sag vertical curve	I	V	II	Front
Flat terrain curve	I	VI	II	Front
Uphill curve	III	III	III	Front
Downhill curve	III	III	II	Rear
Curve in crest vertical curve	III	III	III	Front
Curve in sag vertical curve	V	V	II	Front
Flat terrain spiral transition curve (P|| Sfav)*	I	VI	II	Front
Flat terrain spiral transition curve (P|| Fav)*	I	I	II	Front
Uphill spiral transition curve (P|| Sfav)	III	III	III	Front
Uphill spiral transition curve (P|| Fav)	III	III	III	Front
Downhill spiral transition curve (P|| Sfav)	III	III	II	Rear
Downhill spiral transition curve (P|| Fav)	III	I	II	Rear
Spiral transition curve in crest vertical curve (P|| Sfav)	III	III	III	Front
Spiral transition curve in crest vertical curve (P|| Fav)	III	III	III	Front
Spiral transition curve in sag vertical curve (P||Sfav)	I	I	II	Front
Spiral transition curve in sag vertical curve (P||Fav)	I	I	II	Front

*("P || Fav" and "P|| sfav" stand for the favourable and adverse conditions of the same horizontal-vertical alignment)

The Friction Diagram can be used for both directions of travel and for both the wet and icy road pavement conditions. The friction coefficient, in this case study, was assumed as dependent on speed, from the diagram reported below, obtained from literature data[14, 15].

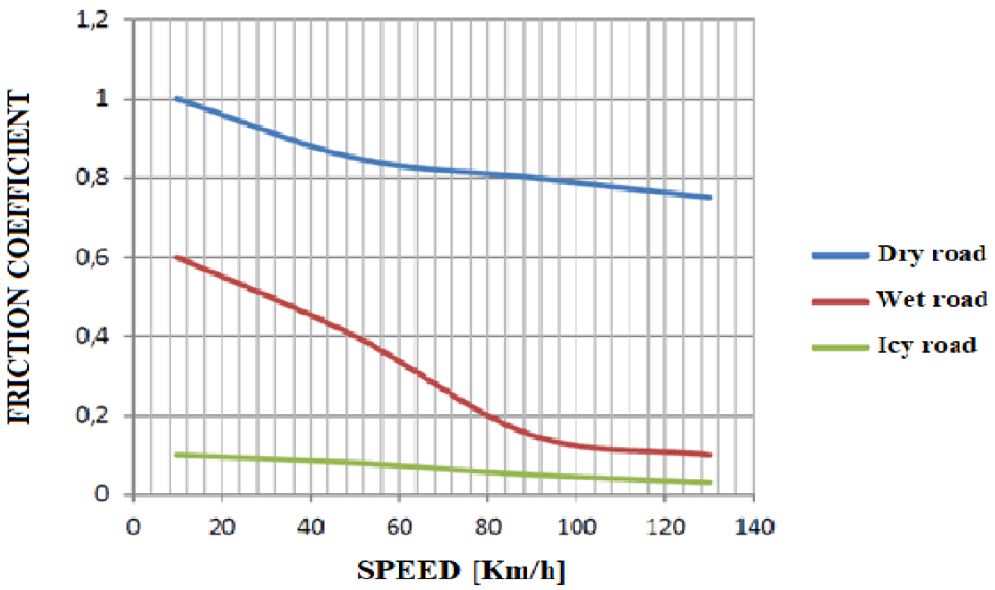

Fig. 13.24: Speed - Friction coefficient diagram.

[14] Lamm R., Psarianos B., Mailaender T. (1999), *Highway design and traffic safety engineering handbook*, McGraw Hill, New York, USA.

[15] Canale S., Leonardi S., Nicosia F. (1996), "Analisi critica del fenomeno dell'aderenza in campo stradale e ferroviario", *quaderno 88*, Istituto Strade, Ferrovie Aeroporti, Facoltà di Ingegneria, Università degli studi di Catania.

The friction diagram is then the graphical representation of the Friction Used for each cross section and it could be drawn together with the speed profile on the elevation profile.

The Friction Diagram could be used as a diagnostic tool to highlight unsafe road sections. The unsafe sections are the ones where the Friction Used value is close or greater than 100%.

Instead, in the design phase, the Friction Diagram is a useful tool to correctly design the vertical-horizontal alignment in order to ensure that the Friction Used is always less than 100%. The friction diagram is obtained for each direction of travel, for wet and icy road conditions, for each cross section, considering each critical vehicle, and for each horizontal-vertical alignment combination.

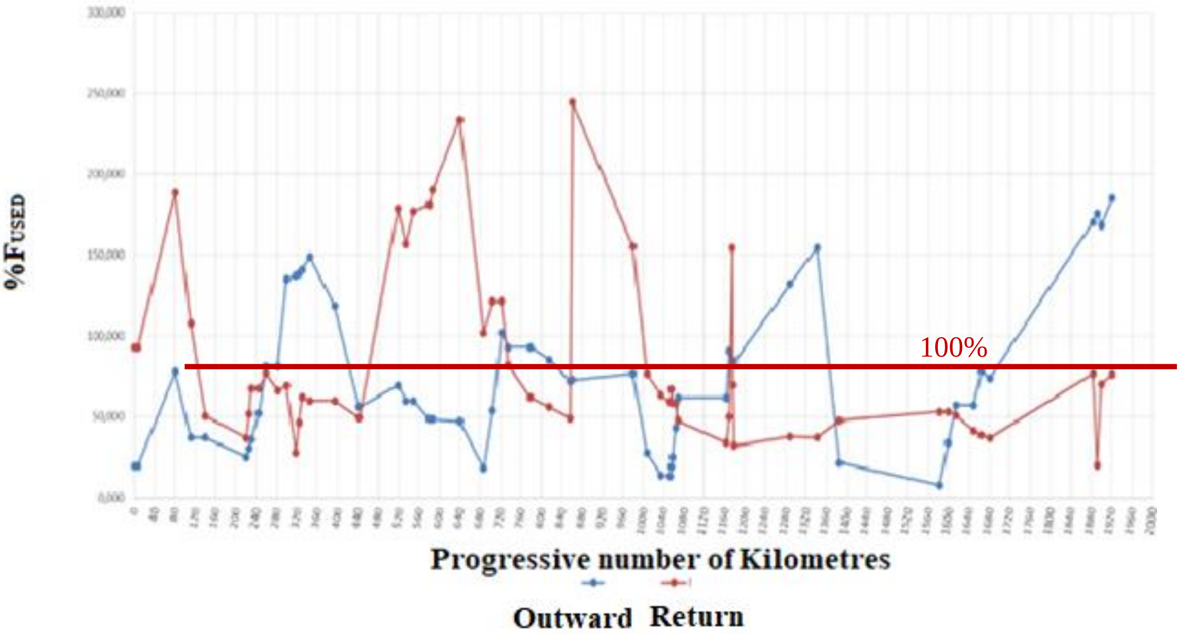

Fig. 13.25: Friction diagram, in case of wet roads (Colonna et al., 2018[5]).

It is helpful remarking that the friction diagram is a "theoretical diagram" which is far from the actual driver behaviours and it does not take into account the environmental variables (such as the vertical alignment). Using the operating speed, for both the directions of travel and for each section, could have brought to more realistic results for the actual Friction Used.

The remarks about the sections, in the outward direction, where the Friction Used is greater than 100% (wet conditions) are explained as follows:

- cross sections 12-14 (300-323 m), the critical vehicle III, speeds varying from 71.34 Km/h to 74.26 Km/h, in the acceleration phase, friction coefficient from 0.257 to 0.237; the vehicle faces the spiral curve at the exit from Curve 1 with cross slope varying from 6.4% to -2.5% on the following tangent (the spiral curve parameters are not compliant with regulations);
- cross sections 15-16 (330-345 m), critical vehicle III, speeds varying from 75.03 Km/h to 76.89 Km/h while accelerating, friction coefficient varying from 0.232 to 0.220; the vehicle faces an uphill segment whose longitudinal slope is 4.9%. The driving force needed for accelerating the vehicle, weighting 1195.31 Kg, equal to 0.8 m/s^2 is the cause of the high value of Friction Used;
- cross section 17 (394 m), critical vehicle III, speed equal to 82.84 Km/h while accelerating, friction coefficient equal to 0.183. The vehicle is on the top of the crest with a Δi of 9.93% and a vertical Radius, R_V equal to 997.20 m. At the crest, the vehicle weight is lower thanks to the lifting centrifugal force while accelerating, so the adherent weight is reduced as well;
- cross section 27 (721 m), critical vehicle III, constant speed of 84.79 Km/h and friction coefficient 0.173; the vehicle faces the Curve 2, whose radius R is 290 m where there is a crest too. The radius of the crest curve is 2064 m. So the centrifugal forces act together. The %F_{USED} obtained by the available data is slightly greater than 100%;
- cross section 46 (1288 m), critical vehicle III, speed equal to 83.88 Km/h, friction coefficient 0.178 while accelerating, The vehicle drives on an uphill road whose longitudinal slope is 1.20%. The cause of the great %F_{USED} is the high mass of the vehicle, 1195.3 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;

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- cross section 47 (1342 m), critical vehicle V, with a speed of 90.88 Km/h and friction coefficient equal to 0.147 while accelerating. The vehicle faces a sag curve between two grades. The vertical radius of the sag curve is 4606 m. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 3515.63 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;
- cross section 55-56-57-58 (1884-1922 m), critical vehicle III, speed varying from 85.35 Km/h to 90.93 Km/h, friction coefficient ranging from 0.170 to 0.146 while accelerating. The vehicle is on an uphill road whose longitudinal slope varies from 3.32% to 2.40%, travelling on a vertical crest curve (R_V equal to 1777.49 m). The vehicle drives on an uphill road whose longitudinal slope is 1.20%. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 1195.3 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope.

The remarks about the sections, in the return direction, where the Friction Used is greater than 100% are explained as follows:

- cross section 16 (1342 m), critical vehicle V for a speed equal to 90.88 Km/h while accelerating and friction coefficient of 0.147. The vehicle travels in a sag curve between two downhill segments whose radius R_V is 4606 m. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 3515.63 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;
- cross section 26-27 (980-860 m), critical vehicle III, speed varying between 78.60 Km/h and 94.29 Km/h and friction coefficient ranging from 0.206 to 0.136 while accelerating; the vehicle faces an uphill segment whose longitudinal slope is 4.60%. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 1195.3 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;
- cross section 32-33 (778-734 m), critical vehicle, constant speed of 84.79 Km/h and friction coefficient of 0.173. The vehicle enters in the Curve 2 (Radius R equal to 290 m) placed on an uphill segment, whose slope is 4.60%. The cross slope of the curve is 5.60%. The friction used is always greater than 100%, even assuming the cross slope equal to 7%;
- cross section 34 (721 m), critical vehicle III, constant speed of 84.79 Km/h and friction coefficient of 0.173. The vehicle faces the Curve 2 in the beginning part of the crest curve whose radius is 2063.79m, under the combined effect of two centrifugal forces. The friction used in this section is lower than the one used in the cross sections 32 and 33. This suggests that the grade contribution to the Friction Demand is greater than the contribution required by the centrifugal force of the vertical curve;
- cross sections from 35 to 37 (708-691 m), critical vehicle III, speed ranging between 86.21 Km/h and 88.15 Km/h and friction coefficient varying from 0.166 to 0.157 while accelerating. The vehicle on the exit of the Curve 2 faces the spiral transition curve on a crest curve, with cross slope varying from 5.60% to -2.50%;
- cross section 38 (687 m), critical vehicle III, speed equal to 88.65 Km/h and friction coefficient to 0.155 while accelerating. The vehicle is on a tangent at the end of the crest curve (R_V equal to 2063.79 m). The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 1195.3 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;
- cross sections 39 and 40 (640-586), critical vehicle III, speed ranging between 93.97 Km/h and 100 Km/h and the friction coefficient from 0.137 to 0.127 while accelerating. The vehicle faces an uphill tangent segment. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 1195.3 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;
- cross section 55 (111 m), critical vehicle V, speed equal to 87.33 Km/h and friction coefficient to 0.161 while accelerating. The vehicle faces a sag whose R_V is 783.89 m. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 3515.63 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope;
- cross section 56 (81 m), critical vehicle III, speed 90.95 Km/h and friction coefficient equal to 0.146 while accelerating. The vehicle faces an uphill road whose longitudinal slope is 2.60%. The cause of the great $\%F_{USED}$ is the high mass of the vehicle, 1195.3 Kg which needs a great driving force to accelerate (0.8 m/s^2) on the slope.

It is blatant from the analyses of the causes of the friction losses that the possible countermeasures to mitigate this effect may consist in adjusting the horizontal-vertical alignment or reducing speeds. In all the curves, crashes occurred, considering the reconstructed design speed and a friction used always less than 100%. Hence, the causes of the crashes occurred in the curves could be related to the high speeds rather than to the friction loss. By the way, the friction loss must be taken into account, because it is likely to affect the crash occurrence and the severity of crashes. The only way to increase the skid resistance with the speeds from the speed diagram, ensuring friction used always less than 100% is to deeply modify the horizontal-vertical alignment. However, this solution implies changing the entire road layout with high costs associated and maybe not affordable for the road agencies. Re-designing the road layout should be considered as an alternative only in the following cases:

- if the road segment has a strategic function in the entire road network (for high traffic volumes, important role in the network);
- every time a new road segment is being designed.

In this case, considering the territorial function of the road segment and the environmental constraints of the analysed landscape, a possible solution to increase the skid resistance is acting on the speed, reducing it. This reduction could be achieved by adequate posted speed limits, identifying solutions and countermeasures to ensure the respect of the posted limits (speed control).

After a trial and error procedure, reducing iteratively the posted speed, the speed limit was reduced from 100 Km/h (used for the speed diagram) to 60 Km/h. This speed is determined in order to have the Fused value from the friction diagram in wet conditions always below 100 %. The maximum value of the posted speed limit which can be used to respect this condition can be considered as the "safe speed", complying with both requirements from standards and to avoid friction-related skidding (Colonna et al., 2017[16]). The threshold can be even lowered for including a safety margin (e.g., having the Fused always below 80%).

In the icy conditions, friction is lost (Fused greater than 100 %) in most of the sections even with a posted speed of 60 Km/h. This problem can be mitigated by using snow tyres or slowing down to a speed dramatically lower than 60 Km/h.

It is notable that the friction diagrams calculated with speeds taken from the speed profile and from safe speeds are not proportional. In fact, speeds, accelerations and decelerations vary with the specific critical vehicle determined for each section, which may vary as well according to the boundary conditions.

13.5.5 Haddon matrix

Often, reasons for crashes are attributed to one "motivation"; actually, road crashes are the result of a series of events influenced by various factors that contribute to their occurrence (time of the day, driver attention, speed, vehicle condition, road design, etc.). These factors influence the sequence of events, before, during and after the crash.

- <u>Before the crash</u>: contributing to increase/decrease the probability of the crash to occur (e.g. state of brakes);
- <u>during the crash</u>: contributing to increase/decrease the crash severity (e.g. airbags);
- <u>after the crash</u>: contributing to increase/decrease damages and injuries (e.g. rescue).

It is possible to outline factors that mainly influence the crash occurrence and severity in the following macro categories[1,17]:

- <u>human</u>: age, driving capability and experience, consciousness, attention, fatigue, sobriety, etc.;
- <u>vehicle</u>: design, construction and maintenance;
- <u>infrastructure/environment/traffic (road-related)</u>: geometric design, cross-section, traffic control, friction, slope, signs, weather conditions, sight distance, etc.

The Haddon Matrix[17,1] is a synthetic tool that links road crash events with crash factor categories.

13.5.5.1 Reconstruction of Haddon matrices for each crash

The following tables show possible examples of Haddon's matrices for each crash that occurred along the segment under investigation. These tables are populated with possible factors which may influence crashes, for illustrative purposes.

[16] Colonna P., Berloco N., Intini P., Ranieri V. (2017), "The method of the friction diagram: New developments and possible applications", In *Transport Infrastructure and Systems: Proceedings of the AIIT International Congress on Transport Infrastructure and Systems*, 309 (Rome, Italy, 10-12 April), CRC Press., Cleveland, USA.

[17] Haddon W. (1972), "A logical framework for categorizing highway safety phenomena and activity", *Journal of Trauma and Acute Care Surgery*, 12(3), 193-207.

Tab. 13.26: Haddon Matrix - crash n. 1.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, bad curve evaluation	Probably smooth tires	Wet pavement with probable presence of mud, inadequate curve
During the crash (causes of crash severity)	Seatbelt not fastened	Airbag functionality	Friction and transverse slope of the road, guard rail inadequate
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, follow-up medical care

Tab. 13.27: Haddon Matrix: crash n. 2.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, bad curve evaluation	Probably smooth tires	Wet pavement, inadequate curve
During the crash (causes of crash severity)	Seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, follow-up medical care

Tab. 13.28: Haddon Matrix: crash n. 3.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation	Probably smooth tires	Icy pavement, inadequate curve
During the crash (causes of crash severity)	Vulnerability to injury, age, gender	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse and longitudinal slope of the road
After the crash (causes of crash outcome)	Age, gender		

Tab. 13.29: Haddon Matrix: crash n. 4.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation, tiredness	Probably smooth tires	Wet pavement, likely glare from the sun before sunset, inappropriate curve
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, follow-up medical care

Tab. 13.30: Haddon Matrix: crash n. 5.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad evaluation, tiredness	Probably smooth tires	Wet pavement, likely glare from the sun before sunset, inappropriate curve
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, follow-up medical care

Tab. 13.31: Haddon Matrix: crash n. 6.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation, tiredness	Probably smooth tires	Wet pavement, inadequate curve
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened		Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender		

Tab. 13.32: Haddon Matrix: crash n. 7.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation, tiredness	Probably smooth tires	Wet pavement, inadequate curve
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

Tab. 13.33: Haddon Matrix: crash n. 8.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation, tiredness	Probably smooth tires	Wet pavement
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender		

Tab. 13.34: Haddon Matrix: crash n. 9.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age	Probably smooth tires	Wet pavement
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

Tab. 13.35: Haddon Matrix: crash n. 10.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation	Probably smooth tires	Wet pavement, likely glare from the sun before sunset, inappropriate curve
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

Tab. 13.36: Haddon Matrix: crash n. 11.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation, tiredness	Probably smooth tires	Wet pavement
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

Tab. 13.37: Haddon Matrix: crash n. 12.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, tiredness	Probably smooth tires	Debris pavement
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and transverse slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

Tab. 13.38: Haddon Matrix: crash n. 13.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad evaluation	Probably smooth tires	Wet pavement
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle's ability to absorb impact energy	Friction and slope of the pavement with probable presence of mud
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

Tab. 13.39: Haddon Matrix: crash n. 14.

Period	*Human factors*	*Vehicle factors*	*Road factors*
Before the crash (danger causes)	Distracted driving, age, bad curve evaluation	Probably smooth tires	Wet pavement
During the crash (causes of crash severity)	Vulnerability to injury, age, seatbelt not fastened	Airbag functionality, vehicle ability to absorb impact energy	Friction and slope of the road
After the crash (causes of crash outcome)	Age, gender	Easy removal of injured passengers	Time and quality of emergency response, medical care

13.5.6 Possible countermeasures for each homogeneous segment

Previously, the section under investigation was divided into eleven homogeneous segments based on the road geometric characteristics. In order to optimize countermeasures, other factors that identify the boundaries of homogeneous segments should be considered:

- functional road class (considering possible further divisions within the class according to different cross-sectional arrangements, such as the number of lanes);
- context and environmental characteristics (e.g. flat or mountain section);
- traffic (volumes, traffic components, density, time variability, etc.).

In this case, the road functional class is homogeneous along the two analysed kilometres, as well as the environmental context. In addition, the traffic has been assumed to be constant because there are no major road intersections in the segment that could cause significant AADT variations. For these reasons, the previously reported classification is valid for the countermeasure analysis.

Countermeasures for each homogeneous segment are proposed below, according to the specific crash type to be reduced. Fatal and injury crashes are in bold. Countermeasures were chosen according to the critical issues identified in the diagnosis process. Factors, apparently not related to observed crashes, were also considered: loss of friction in different parts of the segment and lack of sight distance at some accesses.

13.5.6.1 Homogeneous segment 2 - Curve 1

Data:

- curve radius: 170.00 m;
- cross slope: 6.73%;
- composite length (curve + transition curves): 108.85 m.

Problems detected by the analysis of:

- crash data;
- study of the horizontal-vertical alignment;
- speed profile;
- friction diagram;
- collision diagrams;
- condition diagrams;
- inspections.

Types of problems found:

- crash n. 1: fatal skidding-related crash in wet conditions;

- lane width: 3.00 m;
- shoulder width: 0.50 m;
- absence of edge line markers;
- speed difference between tangents 1 and 2 and with the speed at curve 1 not compliant with standards;
- kick-back not compliant with standards - transition curve 1sx;
- kick-back not compliant with standards - transition curve 1dx;
- drainage ditch with mud on it.

Problem analysis and solutions

As far as the existing conditions are concerned, some measures are analysed related to:

- speed;

the design speed in the tangent 2 is 100 km/h, the design speed in the curve 1 is 69.19 km/h. The difference is greater than 10 km/h.

Solution. Two alternative solutions are available to mitigate this problem:

1) redesigning the curve with a radius of at least 400 m. This would partly solve the problem. For sight distance problems, the tangent speed should still be limited to 80 km/h;
2) automatic speed control.

- horizontal alignment;

the radius of the curve must be at least 400 m. Transition curves will also have to be redesigned. In addition, the width of lanes and shoulders is not adequate.

Solutions:

1) redesigning the curve with a radius of at least 400 m;
2) redesigning transition curves with a value of A_{min} = 182.00 m;
3) lane widening;
4) shoulder widening.

- Edge line markers;

throughout the entire road, margin markers are either damaged or absent.

Solution:

1) reset/installation of edge line or road markers, every 5 m.

- Safety devices.

In the fatal crash n. 1, the impact damaged the “H1” type barrier near a 2 m high embankment.

Solution:

1) installation of a “H2” barrier for bridges.

Short term adjustment interventions:

- increase in cross slope from 6.7% to 7%;
- restore/install edgeline markers such as road markers and rumble strips;
- enhance barriers at Km 6.200;
- automatic speed control.

Long term adjustment interventions:

- shoulder widening from 0.50 m to 1.25 m;
- lane widening from 3.00 m to 3.50 m;
- curve redesign with R ≥ 400 m, including the associated transition curves;
- curve widening and shifting of dry-stone walls with any trees included, to increase sight distance.

Maintenance interventions:

- Drainage ditch maintenance.

13.5.6.2 Homogeneous Segment 2 - Tangent 2

Data:

- length: 36.96 m;
- cross slope: 2.50%.

Problems detected by the analysis of:

- crash data;
- study of the horizontal-vertical alignment;

- speed profile;
- friction diagram;
- collision diagrams;
- condition diagrams;
- inspections.

Types of problems found:
- crash n. 13: fatal skidding-related crash in wet conditions;
- lane width: 3.00 m;
- shoulder width: 0.50 m;
- absence of edge line markers;
- speed difference between tangents 1 and 2 and with the speed at curve 1 not compliant with standards;
- drainage ditch with mud on it.

Problem analysis and solutions

As far as the existing conditions are concerned, some measures are analysed related to:
- speed;

  the speed in the tangent 2 is 100 km/h, the speed in the curve 1 is 69.19 km/h. The difference is greater than 10 km/h.

  Solution: two alternative solutions are available to mitigate this problem:

  1) redesigning the curve with a radius of at least 400 m. This would partly solve the problem. For sight distance problems, the tangent speed should be limited to 80 km/h;
  2) automatic speed control.
- horizontal alignment;

  the width of lanes and shoulders is not adequate.

  Solution:

  1) lane widening;
  2) shoulder widening;
- edge line markers.

  Throughout the entire road, edge line markers are either damaged or absent.

  Solution: reset/installation of edge line markers or road markers, every 10 m.

Short term adjustment interventions:
- restore/install edge line markers and rumble strips;
- automatic speed control.

Long term adjustment interventions:
- shoulder widening from 0.50 m to 1.25 m;
- lane widening from 3.00 m to 3.50 m;
- increase the radius of the vertical curve.

Maintenance interventions:
- drainage ditch maintenance;
- improve the removal of rainwater (and mud), also coming from the two accesses at Km 6.5.

13.5.6.3 Homogeneous Segment 7 - Curve 3

Data:
- curve radius: 160.00 m;
- cross slope: 6.20%;
- composite length (curve + transition curves): 126.20 m.

Problems detected by the analysis of:
- crash data;
- study of the horizontal-vertical alignment;
- speed profile;
- friction diagram;
- collision diagrams;
- condition diagrams;

- inspections.

Types of problems found

- crash n. 2, 4, 5, 6, 7, 8, 10, 14: fatal skidding-related crash in wet conditions;
- lane width: 3.00 m;
- shoulder width: 0.50 m;
- absence of edge line markers;
- ratio between curve radius 3 and the previous/next curve radii not compliant with standards;
- kick-back - transition curve 1sx not compliant with standards;
- kick-back - transition curve 1dx not compliant with standards;
- curve placed immediately after the end of the vertical curve (sag);
- low optical contrast between the curve and the surrounding environment;
- gravel in the drainage ditch.

Problem analysis and solutions

As far as the existing conditions are concerned, some measures are analysed related to:

- horizontal alignment;
  curve and transition curves should also be redesigned. In addition, the width of lanes and shoulders is not adequate.
  Solution:
  1) redesign of the curve with a radius of at least 350 m;
  2) redesign of transition curves with a value of A_{min} = 184.40;
  3) lane widening;
  4) shoulder widening;
- vertical alignment;
  curve 3 is placed immediately after the end of the vertical curve (sag) included in part of the tangent 3. In this way, the driver has an erroneous perception of road edges.
  Solution: redesign of grades;
- edge line markers;
  throughout the entire road, edge line markers are either damaged or absent.
  Solution: reset/installation of edge line markers or road markers, every 5 m.

Short term adjustment interventions:

- planting trees (e.g., oaks) on the outside of the curve in order to improve the optical perception of the road;
- cutting/placing trees placed on the roadside of the internal part of the curve;
- installation of curve warning signs (with flashing lights);
- increase in cross slope from 6.7% to 7%;
- restore/install edge line markers and rumble strips;
- automatic speed control.

Long term adjustment interventions:

- shoulder widening from 0.50 m to 1.25 m;
- lane widening from 3.00 m to 3.50 m;
- curve redesign with R ≥ 400m, with associated transition curves;
- curve widening and shifting of dry-stone walls with any trees included, to increase sight distance.
- redesign of the curve with a radius of at least 350 m including the associated transition curves (the increase in radius would, however, interfere with the vertical curve placed immediately before the curve);
- increase of the vertical curve radius (it could interfere with the horizontal curve, but it would increase the ratio between R_v and R);
- curve widening and shifting of dry-stone walls with any trees included, to increase sight distance.

Maintenance interventions:

- drainage ditch maintenance with possible reinforced concrete coating.

13.5.6.4 Homogeneous Segment 10 - Curve 4

Data:

- curve radius: 80.00 m;

- cross slope: 6.46%;
- composite length (curve + transition curves): 98.18 m.

Problems detected by the analysis of:

- crash data;
- study of the horizontal-vertical alignment;
- speed profile;
- friction diagram;
- collision diagrams;
- condition diagrams;
- inspections.

Types of problems found:

- crash n. 3, 11: fatal skidding-related crash in wet conditions;
- lane width: 3.00 m;
- shoulder width: 0.50 m;
- absence of edge line markers;
- speed difference between tangents 4 and 5 not compliant with standards;
- ratio between radius of curves 4 and 3 not compliant with standards;
- radius of the curve lower than the minimum provided by standards;
- kick-back - transition curve 1sx not compliant with standards;
- kick-back - transition curve 1dx not compliant with standards;
- low optical contrast between the curve and the surrounding environment.

Problem analysis and solutions

As far as the existing conditions are concerned, some measures are analysed related to:

- speed;

  the speed in the tangent 5 is 100 km/h, the speed in the curve 4 is 55.73 km/h. The difference is greater than 10 km/h.

  Solution: two alternative solutions are available to mitigate this problem:
  1) redesigning the curve with a radius of at least 400 m. This would partly solve the problem. For sight distance problems, the tangent speed should be limited to 80 km/h;
  2) Automatic speed control;
- horizontal alignment;

  the radius of the curve must be at least 400 m. Transition curves will also have to be redesigned. In addition, the width of lanes and shoulders is not adequate.

  Solution:
  1) redesign of the curve with a radius of at least 400 m;
  2) redesign of transition curves with a value of A_{min} = 182.00;
  3) lane widening;
  4) shoulder widening;
- friction;

  the pavement in this section is a traditional friction course, not porous; it is partially affected by some hollows and cracks.

  Solution: replacement of the surface course with Antiskid friction course to improve friction;
- edge line markers;

  throughout the entire road, edge line markers are either damaged or absent.

  Solution: reset/install edge line markers or road markers, every 5 m.
- safety devices;

  the barrier (“H1” type, near to 3 m high embankment) close to the external part of the curve is damaged and, in some cases, not anchored to the retaining wall on which it is placed.

  Solution: installation of a type “H2” barrier for bridges.

Short term adjustment interventions:

- friction increase;
- planting trees (e.g., oaks) on the external part of the curve in order to improve the optical perception of the road;
- installation of curve warnings signs (with flashing lights);

- enhance the road barrier at Km 7.7;
- restore/install edge line markers and rumble strips;
- automatic speed control.

Long term adjustment interventions:
- shoulder widening from 0.50 m to 1.25 m;
- lane widening from 3.00 m to 3.50 m;
- increase in cross slope from 6.46% to 7%;
- curve redesign with R ≥ 400m, with associated transition curves.

13.6 Selection of countermeasures

At a given site, the benefit of a countermeasure is proportional to the difference between the expected number of crashes before and after the implementation of a countermeasure or set of countermeasures. For the estimation of expected crashes, the HSM manual, 2010[1] proposes the Empirical-Bayesian Method that allows to calculate the Average Expected Crash Frequency, $N_{Expected}$ (for different severity levels). Since the split of crashes into fatal, injury crashes and property damage-only crashes is not known, the crash severity is supposed. The EB method combines the Observed Crash Frequency (number of crashes actually occurred during the observation period on the segment) of a given site with the Average Expected Crash Frequency (number of crashes per year expected on that site, calculated using a statistical regression model).

The combination of two frequencies is used to limit errors related to both methods. The Observed Crash Frequency is biased because the crash is a random event, therefore a long observation period is needed for its effective estimation. Having short observation periods available, the regression-to-the-mean error occurs. The Predicted Crash Frequency, instead (referring to a model developed for a specific site type and for specific geometric and traffic conditions) is affected by variations in road conditions, both in terms of degradation of the infrastructure and of traffic volume variations. The latter should, therefore, preferably be referred to short observation periods.

According to the HSM manual (2010)[1], the Average Expected Crash Frequency is obtained through the SPF (Safety Performance Function) which represents the average variation in the number of crashes as the traffic volume changes, for a specific type of site.

In this case, the two-way two-lane HSM SPF is the following:

$$N_{SPF} = constant\ x\ AADT \times L \times 365 \times 10^{-6} \qquad \text{(Eq. 13-21)}$$

where:
- N_{SPF} is the Average Predicted Crash Frequency calculated for baseline conditions of all the other variables [number of crashes/years];
- $AADT$ is the Average Annual Daily Traffic in a given section;
- L is the homogeneus segment length [mile].

However, this function does not take into account differences between sites (e.g. geometric) and socio-economic or drivers' cultural differences between countries or regions. For this reason, the formula of the Predicted Average Frequency is equal to:

$$N_{Predicted} = N_{SPF} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) \times C_C \qquad \text{(Eq. 13-22)}$$

Where:

$N_{Predicted}$ is the Average Predicted Crash Frequency for a specific year and for a given site;

$CMF_{1x} \times \ldots \times CMF_{yx}$ is the product of all the Crash Modification Factors based on the actual characteristics of the given site with respect to baseline conditions;

C_C is the calibration coefficient, which takes into account local conditions.

13.6.1 CMFs of possible countermeasures

For the estimation of CMFs, the equations/factors reported in the Chapter 10 of the HSM manual (2010)[1] were used. If an HSM CMF was not available for a given countermeasure, additional CMFs were retrieved in the "CMF Clearinghouse" website.

In this study, the calculation/estimation of the CMFs was performed for each homogeneous segment, distinguishing curves from tangents.

- The CMFs used in this application which were taken from the HSM are: lane width (CMF = 1.17, corresponding in this case to an about 3 m lane and the specific AADT considered, with respect to the baseline lanes considered in the HSM manual, 2010[1]);
- shoulder width and type (CMF = 1.17, corresponding in this case to an about 0.60 m paved shoulder and the specific AADT considered, with respect to the baseline shoulders considered in the HSM);
- horizontal curves (variable CMF according to the curve radius, length and spiral transition curve presence);
- superelevation variance (variable CMF according to the deviation of the superelevation in curve with respect to the required cross slope);
- grades (variable CMF depending on the longitudinal slope);
- driveway density (variable CMF according to the number of accesses per mile and the specific AADT);
- roadside hazard rating (variable CMF according to the qualitative judgement of the roadside hazard conditions).

The HSM CMFs for centreline rumble strips and automatic speed control were not considered in the current conditions, since they are not present on the investigated segment.

The presence/absence of the curve warning signs was considered by using the following CMF values:

- CMF = 1 in baseline conditions;
- CMF = 0.564 in case of presence of curve warning signs (Montella, 2009[18]).

This CMF was retrieved in the "CMF Clearinghouse" website. Additional CMFs used to model specific countermeasures will be reported in the following.

The application of CMFs for all the homogeneous segments investigated has led to the following estimations of predicted crash frequencies:

Tab. 13.40: Total number of predicted crashes for each homogenous segment.

Segment ID	*Npredicted (Total) (crashes/year)*	*Npredicted (fatal + injury) (crashes/year)*	*Npredicted (PDO) (crashes/year)*
Segment 1	0.312	0.100	0.212
Segment 2	0.493	0.158	0.335
Segment 3	0.343	0.110	0.233
Segment 4	0.259	0.083	0.176
Segment 5	0.471	0.151	0.320
Segment 6	0.261	0.084	0.177
Segment 7	0.283	0.091	0.192
Segment 8	0.260	0.083	0.177
Segment 9	0.358	0.115	0.243
Segment 10	0.660	0.212	0.448
Segment 11	0.336	0.108	0.228

Crash frequency can be determined either for Total crashes (A, according to the HSM KABCO scale), or it can be divided into Fatal and Injury crashes (FI) and crashes with Property Damage Only (PDO). HSM default percentages were used to classify crashes into severity classes, since local studies were unavailable. The calibration coefficient Cc was set to 1.26, according to Colonna et al. (2016)[2] for AADT < 10.000 (Puglia region).

The calculation of the predicted crash frequencies for each homogeneous road segment was made with the help of the spreadsheets available at the website: http://www.highwaysafetymanual.org/Pages/Tools.aspx.

Once all the $N_{Predicted}$ of various segments were calculated, it is possible to estimate the $N_{Expected}$ of the current situation, for each homogeneous segment. The total number of observed crashes available is 14: 11 are fatal and injury crashes, and 3 are crashes with property damage only (PDO).

However, the number of PDO crashes is not reliable because when they occur, law enforcement agencies could be often not involved, and the crash is not recorded. Using American statistics, it can be estimated that, if 11 fatal+injury crashes were recorded, an average of at least 23 PDO crashes would have been collected.

Therefore, the $N_{Expected}$ for fatal and injury crashes is calculated and then, it is converted it into the total $N_{Expected}$ through the proportion of crashes, according to American HSM default percentages. The equivalence of fatal +

[18] Montella A. (2009), "Safety evaluation of curve delineation improvements: empirical Bayes observational before-and-after study", *Transportation research record, Journal of the Transportation Research Board*, 2103(1), 69-79.

injury observed crashes with the KABC category according to the HSM severity scale (including possible injuries "C") was therefore assumed because data, obtained by local authorities, were considered more reliable and detailed than those obtained from aggregate data.

The formula used for the calculation of $N_{Expected}$ is as follows:

$$N_{Expected} = w \times N_{Predicted} + (1 - w) \times N_{Observed} \qquad \text{(Eq. 13-23)}$$

where:

- $N_{Expected}$ is the average expected crash frequency for a specific grade of severity (A, FI or PDO);
- $N_{Observed}$ is the number of crashes on the segment under investigation in the observation period (7 years);
- $N_{Predicted}$ is the sum of each annual $N_{predicted}$ in the observation period (7 years). If the traffic would have varied during the observation period, the annual $N_{Predicted}$ would also vary (in this case, the traffic is assumed constant). For this reason, the calculated $N_{predicted}$ has been multiplied by the observation period (7 years).
- w is the factor that weights the predicted crash frequency in the calculation of the expected frequency. The formula proposed by the HSM (2010)[1] for the calculation of w is a function of the sum of the $N_{Predicted}$ over the observation period and not referred to one year:

$$w = \frac{1}{1+K*\sum N_{Predicted}} \qquad \text{(Eq. 13-24)}$$

- k is the local over-dispersion parameter; the parameter provided by the HSM (converted in km) for two-way two-lane rural roads, is equal to:

$$k = \frac{0,38}{L\,[km]} \qquad \text{(Eq. 13-25)}$$

At this point, $N_{Expected}$ (FI) is converted into $N_{Expected}$ (A) and then broken down again for each single year, dividing the data by the observation period. The results are shown in the following Table 13.41.

Tab. 13.41: Summary of EB Method Results.

Segment ID	*Nexpected (Total) (Over 7 years)*	*Nexpected (fatal + injury) (Over 7 years)*	*Nexpected (Total) (crashes/year)*
Segment 1 - Tangent 1	0.480	0.154	0.0690
Segment 2 - Curve 1	5.670	1.820	0.810
Segment 3 - Tangent 2a	0.490	0.157	0.070
Segment 4 - Tangent 2b	0.459	0.147	0.066
Segment 5 - Curve 2	0.543	0.174	0.078
Segment 6 - Tangent 3	5.104	1.638	0.729
Segment 7 - Curve 3	14.729	4.728	2.104
Segment 8 - Tangent 4a	0.461	0.148	0.066
Segment 9 - Tangent 4b	0.614	0.197	0.088
Segment 10 - Curve 4	3.292	1.057	0.470
Segment 11 - Tangent 5	0.602	0.193	0.086
TOTAL	32.444	10.413	4.636

Once the Average Expected Crash Frequency $N_{Expected}$ for the observation period has been determined, the Average Expected Crash Frequency for the future period is calculated. Each of the countermeasures reported at the beginning of this paragraph represents a possible improvement to be made on the road.

13.6.2 CMFs for possible sets of countermeasures

In order to continue with the analysis, it is therefore necessary to establish which countermeasures are to be implemented. In this respect, several sets of countermeasures have been hypothesised that can be implemented along the entire road, rather than for each homogeneous segment, for the following reasons:

- most crashes are similar along the segment;
- all crashes occur along curves which, are close and influential on each other.

The sets of countermeasures hypothesised are shown below. It should be noted that some countermeasures are effective for reducing other critical points found on the road layout, even if apparently not related to observed crashes. The speed control, the Antiskid course and the horizontal alignment redesign are, in fact, the main countermeasures that reduce (or eliminate) the friction problems and the stopping distances.

Set A:

- dangerous curve warning signs and white LED markers provided with photovoltaic power supply in curves 1, 3 and 4, in both directions;
- installation of central rumble strips and road markings;
- installation of transverse rumble strips in thermoplastic laminate close to curves;
- installation of road markers along the layout, both on the middle and on the edges of the road;
- planting trees of the type: Quercus Trojana, on the outside of curves 3 and 4.

Set B:

- installation of automatic speed control (gate-type) installed before and after the investigated segment.

Set C:

- replacement of the existing class "H1" restraint system with a class "H2" for bridges at km 6.200 right side, at km 6.650 left side, at km 7.100 left side, at km 7.700 left side along the road towards Noci;
- replacing the traditional asphalt course with Anti-Skid course along curve 4.

Set D:

- geometric redesign of the road segment with
  - lane and shoulder widening up to the standards provided by D.M. 05.11.2001[9] for a type "C2" road;
  - horizontal curves redesign according to the D.M. 05.11.2001[9];
  - possible curve widening to improve sight distance;
  - vertical curves redesign according to the D.M. 05.11.2001[9].

As a part of these countermeasures, it was decided, for obvious economic and environmental reasons, not to change existing tangents. In any case, to ensure sight distance rules, the speed limit imposed must not exceed 80 km/h. The proposed re-design interventions are reported below.

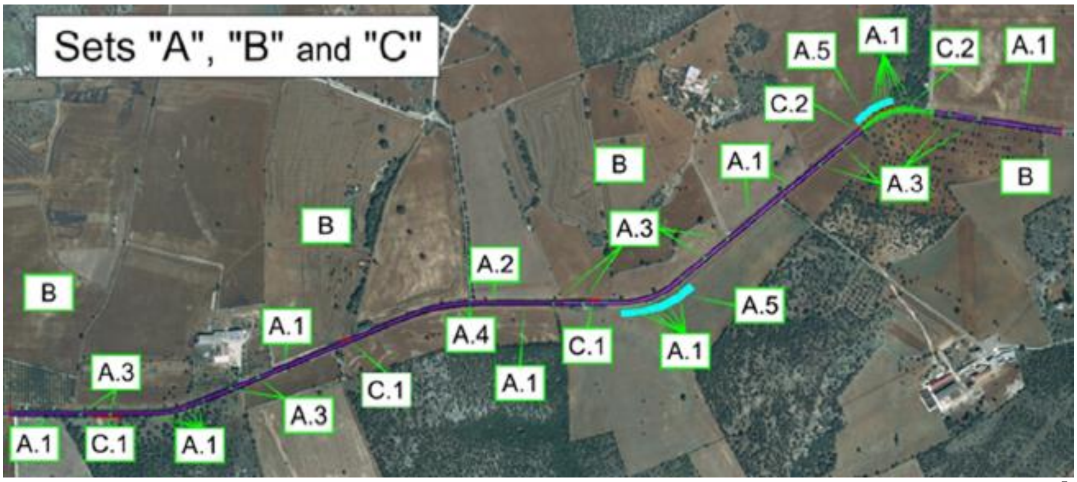

Fig. 13.26: short-term safety measures - sets of countermeasures "A", "B" and "C" (Colonna et al., 2018[5]).

Fig. 13.27: long-term safety measures - set of countermeasures "D" (Colonna et al., 2018[5]).

CMFs assigned to the individual countermeasures identified are reported below.

Tab. 13.42: CMFs assigned to individual countermeasures.

Countermeasure	*CMF*
Curve warning signs (before and at the curve) provided with LED markers[19]	0.518
Centreline Rumble strips[20]	0.940
Road markers (eventually coupled with transverse rumble strips)[20]	0.980
Planting trees for delineating curves[21]	0.980
Automated speed control[22]	0.930
Enhancement of the restraint system[23]	0.994
Improvement of pavement friction[24]	0.852

It should be noted that:

- the CMF relating to the restraint system has been converted into CMF relating to total crashes through the following equation, considering proportion of crashes:

$$CMF\ (A) = CMF\ (FI) \times 0.32 + CMF\ (PDO) \times 0.68 \qquad \text{(Eq. 13-26)}$$

where:

- CMF (PDO) = 1 and CMF (FI) = 0.98. The conversion is necessary because the CMF proposed by the "CMF Clearinghouse" is valid for crashes with severity Fatal, Serious and Minor Injuries;
- the CMF related to the installation of "markers" and "rumble strips" has been converted into total crash CMF through the above equation (with CMF (PDO) = 1 and CMF (FI) = 0.94). The conversion is necessary because the CMF proposed by the "CMF Clearinghouse site" is valid for crashes with severity Fatal, Serious and Minor Injuries;
- the CMF relating to the planting of trees does not come from a scientific study. The effect of curve delineation through trees was equated to that of road markers , because the latter perform the same function. Future research may improve the reliability of this CMF.
- the CMF relating to redesign have been calculated through the application of the HSM CMFs.

Therefore, several possible scenarios have been taken into consideration, analyzing all the possible combinations between the sets of countermeasures A, B and C, and considering the set D as a separate countermeasure. For each set, the future Expected Crash Frequency was then calculated.

The combinations analysed are:

- combination 1: Set A;
- combination 2: Set B;
- combination 3: Set C;
- combination 4: Sets A+B;
- combination 5: Sets A+C;
- combination 6: Sets B+C;
- combination 7: Sets A+B+C;
- combination 8: Set D.

In addition to the scenarios involving the implementation of countermeasures, the scenario in which no safety improvements are made, must be considered.

For the calculation of the crash frequency, it has been assumed that the sets A, B and C have a service life of

[19] Montella A. (2009), "Safety evaluation of curve delineation improvements: empirical Bayes observational before-and-after study", *Transportation research record, Journal of the Transportation Research Board*, 2103(1), 69-79.

[20] Torbic D. J., Hutton J. M., Bokenkroger C. D., Bauer K. M., Harwood D. W., Gilmore D. K., Dunn D. K., Ronchetto J. J., Donnell E. T., Sommer III H. J., Garvey P., Persaud B., Lyon C. (2009), "NCHRP Report 641: Guidance for the Design and Application of Shoulder and Centerline Rumble Strips", *Transportation Research Board,* Washington D. C., USA.

[21] Based on note 16.

[22] Polders E., Daniels S., Hermans E., Brijs T., Wets G. (2014), "To brake or to accelerate? Safety effects of combined speed and red light cameras", *Journal of Safety Research*, Vol. 50, 59-65.

[23] Cafiso S., D'Agostino C., Persaud B. (2014), "Investigating the Influence on Safety of Retrofitting Italian Motorways with Barriers Meeting a New EU Standard", *Transportation Research Board Annual Meeting*, Washington D.C., USA.

[24] Lyon C., Persaud B. (2008), "Safety Effects of a Targeted Skid Resistance Improvement Program", *Texture, Friction Management, and Ride Quality, Proceedings of the 87th Annual Meeting of Transportation Research Board (TRB)*, National Research Council, Washington D. C., USA.

10 years while the set D has a service life of 30 years.

For each combination, it is necessary to recalculate CMFs for each homogeneous segment, as reported below.

Tab. 13.43: CMF - Combination 1.

At-site geometric characteristic

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*		*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6		2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7		2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6		2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6		1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7		3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6		1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6		2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6		1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6		1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6		4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6		1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strips*	*Auto. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	2.080
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	0.94	1.000	0.518	0.980	1.000	1.000	1.000	1.932
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	2.080
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	1.724
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	0.94	1.000	1.000	0.980	1.000	1.000	1.000	2.695
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	1.546
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	0.94	1.000	0.518	0.980	0.980	1.000	1.000	1.694
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	1.723
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	1.637
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	0.94	1.000	0.518	0.980	0.980	1.000	1.000	3.593
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	1.000	1.000	0.980	1.000	1.000	1.000	1.546

Ratio between CMF combination and cmf site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.921	0.769	0.838	0.920	0.837	0.837	0.829	0.921	0.837	0.754	0.837

Once the CMFs have been obtained, it is possible to calculate the $N_{Expected}$ for each combination of sets of countermeasures previously defined, for each homogeneous segment and for each year within the service life. However, since a constant AADT for the entire observation period was considered, the formula used is as follows:

$$N_{Expected,Ci,(A)} = N_{Expected,(A)} \times \left(\frac{N_f}{N_p}\right) \times \left(\frac{CMF_{1f}}{CMF_{1p}}\right) \times \left(\frac{CMF_{2f}}{CMF_{2p}}\right) \times \ldots \times \left(\frac{CMF_{nf}}{CMF_{np}}\right) \qquad \text{(Eq. 13-27)}$$

where:

- $N_{Expected,\ C1,\ (A)}$ is the average expected crash frequency, related to the combination of sets of countermeasures *i* (in this case from 1 to 8), calculated on total crashes and referred to the particular year of service life;
- $N_{Expected,\ (A)}$ is the average expected crash frequency, related to the current state (without countermeasures), calculated on total crashes and referred to the particular year of service life;
- $\frac{N_f}{N_p}$ is the ratio that considers the traffic variation between future and past frequency in base conditions (in this case it is equal to 1);
- $\left(\frac{\mathrm{CMF_{1f}}}{\mathrm{CMF_{1p}}}\right)$ x … x $\left(\frac{\mathrm{CMF_{nf}}}{\mathrm{CMF_{np}}}\right)$ is the ratio between the CMFs in the design conditions (combination *i*) and the CMFs in the current conditions.

In order to obtain the expected average crash frequency, for fatal and injury crashes and crashes with property

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damage only, the total crash frequency has been multiplied by the default HSM proportions. The difference between $N_{Expected,\ Ci}$ and $N_{Expected}$ is a variation of the expected average crash frequency $\Delta N_{Expected}$, related to a particular year of service life, as shown in the following tables.

Tab. 13.44: $\Delta N_{expected}$ - Combination 1.

After 10 years-lifetime

Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.921	0.054	0.010	0.037
Segment 2	0.769	1.247	0.399	0.848
Segment 3	0.838	0.055	0.018	0.038
Segment 4	0.920	0.052	0.017	0.035
Segment 5	0.837	0.061	0.020	0.042
Segment 6	0.837	0.575	0.184	0.391
Segment 7	0.829	3.595	1.150	2.445
Segment 8	0.921	0.052	0.017	0.035
Segment 9	0.837	0.069	0.022	0.047
Segment 10	0.754	0.804	0.257	0.546
Segment 11	0.837	0.068	0.022	0.046

Below are the calculations made to obtain CMF and $N_{Expected}$ for the other combinations.

Tab. 13.45: CMF - Combination 2.

At-site geometric characteristic

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6	2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7	2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6	2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6	1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7	3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6	1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6	2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6	1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6	1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6	4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6	1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strips*	*Auto. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	2.100
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	1.000	0.93	0.564	1.000	1.000	1.000	1.000	2.124
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	2.100
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.741
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	1.000	0.93	1.000	1.000	1.000	1.000	1.000	2.721
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.561
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	1.000	0.93	0.518	1.000	0.980	1.000	1.000	1.710
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.739
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.653
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	1.000	0.93	0.564	1.000	0.980	1.000	1.000	3.949
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.561

Ratio between CMF combination and cmf site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.930	0.845	0.842	0.929	0.845	0.845	0.837	0.929	0.845	0.828	0.845

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Tab. 13.46: $\Delta N_{expected}$ - Combination 2.

After 10 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.930	0.048	0.015	0.033
Segment 2	0.845	0.567	0.181	0.386
Segment 3	0.846	0.049	0.016	0.033
Segment 4	0.929	0.046	0.015	0.031
Segment 5	0.845	0.054	0.017	0.037
Segment 6	0.845	0.510	0.163	0.347
Segment 7	0.837	1.473	0.471	1.002
Segment 8	0.929	0.046	0.015	0.031
Segment 9	0.845	0.061	0.020	0.042
Segment 10	0.828	0.329	0.105	0.224
Segment 11	0.845	0.060	0.019	0.041

Tab. 13.47: CMF - Combination 3.

At-site geometric characteristic

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*		*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6		2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7		2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6		2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6		1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7		3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6		1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6		2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6		1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6		1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6		4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6		1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strips*	*Auto. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	2.258
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	1.000	1.000	0.564	1.000	1.000	0.994	1.000	2.270
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	2.258
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	1.000	1.000	1.000	1.000	1.000	0.994	1.000	1.860
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	1.000	1.000	1.000	1.000	1.000	1.000	1.000	2.926
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.679
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	1.000	1.000	0.564	1.000	1.000	0.994	1.000	2.031
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.870
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.778
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	1.000	1.000	0.564	1.000	1.000	0.994	1.000	4.307
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.679

Ratio between CMF combination and CMF site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
1.000	0.903	0.909	0.993	0.909	0.909	0.993	0.999	0.908	0.903	0.909

Tab. 13.48: $\Delta N_{expected}$ - Combination 3.

After 10 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	1.000	0.000	0.000	0.000
Segment 2	0.903	0.049	0.016	0.033
Segment 3	0.909	0.000	0.000	0.000
Segment 4	0.993	0.004	0.001	0.003
Segment 5	0.909	0.000	0.000	0.000
Segment 6	0.909	0.000	0.000	0.000
Segment 7	0.993	0.126	0.040	0.086
Segment 8	0.999	0.000	0.000	0.000
Segment 9	0.908	0.000	0.000	0.000
Segment 10	0.903	0.720	0.230	0.490
Segment 11	0.909	0.000	0.000	0.000

Tab. 13.49: CMF - Combination 4.

At-site geometric characteristic

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6	2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7	2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6	2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6	1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7	3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6	1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6	2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6	1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6	1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6	4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6	1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strips*	*Autom. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.934
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	0.94	0.93	0.518	0.98	1.000	1.000	1.000	1.797
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.934
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.603
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	0.94	0.93	1.000	0.98	1.000	1.000	1.000	2.507
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.438
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	0.94	0.93	0.518	0.98	0.980	1.000	1.000	1.575
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.602
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.523
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	0.94	0.93	0.518	0.98	0.980	1.000	1.000	3.341
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.438

Ratio between CMF combination and CMF site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.857	0.715	0.779	0.856	0.779	0.779	0.771	0.856	0.778	0.701	0.779

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Tab. 13.50: $\Delta N_{expected}$ - Combination 4.

After 10 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.857	0.098	0.031	0.067
Segment 2	0.715	1.726	0.552	1.174
Segment 3	0.779	0.100	0.032	0.068
Segment 4	0.856	0.094	0.030	0.064
Segment 5	0.779	0.112	0.036	0.076
Segment 6	0.779	1.044	0.334	0.710
Segment 7	0.771	4.816	1.541	3.275
Segment 8	0.856	0.094	0.030	0.064
Segment 9	0.778	0.125	0.040	0.085
Segment 10	0.701	1.076	0.344	0.732
Segment 11	0.779	0.123	0.039	0.084

Tab. 13.51: CMF - Combination 5.

At-site geometric characteristic									
Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*		*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6		2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7		2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6		2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6		1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7		3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6		1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6		2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6		1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6		1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6		4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6		1.847

CMF															
Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strips*	*Autom. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	1.000	1.000	0.98	1.000	1.000	1.000	2.080
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	0.94	1.000	0.518	0.98	1.000	0.994	1.000	1.920
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	1.000	1.000	0.98	1.000	1.000	1.000	2.080
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	0.94	1.000	1.000	0.98	1.000	0.994	1.000	1.714
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	0.94	1.000	1.000	0.98	1.000	1.000	1.000	2.695
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	1.000	1.000	0.98	1.000	1.000	1.000	1.546
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	0.94	1.000	0.518	0.98	0.980	0.994	1.000	1.684
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	0.94	1.000	1.000	0.98	1.000	1.000	1.000	1.723
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	0.94	1.000	1.000	0.98	1.000	1.000	1.000	1.637
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	0.94	1.000	0.518	0.98	0.980	0.994	0.852	3.043
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	1.000	1.000	0.98	1.000	1.000	1.000	1.546

Ratio between CMF combination and CMF site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.921	0.764	0.838	0.915	0.837	0.837	0.824	0.921	0.837	0.638	0.837

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Tab. 13.52: ΔN$_{expected}$ - Combination 5.

After 10 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.921	0.054	0.017	0.037
Segment 2	0.764	1.288	0.412	0.876
Segment 3	0.838	0.056	0.018	0.038
Segment 4	0.915	0.056	0.018	0.038
Segment 5	0.837	0.062	0.020	0.042
Segment 6	0.837	0.575	0.184	0.391
Segment 7	0.824	3.700	1.184	2.516
Segment 8	0.921	0.052	0.017	0.035
Segment 9	0.837	0.069	0.022	0.047
Segment 10	0.638	1.400	0.448	0.952
Segment 11	0.837	0.068	0.022	0.046

Tab. 13.53: CMF - Combination 6.

At-site geometric characteristics

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6	2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7	2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6	2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6	1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7	3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6	1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6	2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6	1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6	1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6	4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6	1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strips*	*Autom. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	2.100
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	1.000	0.93	0.564	1.000	1.000	0.994	1.000	2.111
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	2.100
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	1.000	0.93	1.000	1.000	1.000	0.994	1.000	1.730
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	1.000	0.93	1.000	1.000	1.000	1.000	1.000	2.721
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.561
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	1.000	0.93	0.564	1.000	1.000	0.994	1.000	1.889
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.739
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.653
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	1.000	0.93	0.564	1.000	1.000	0.994	1.000	4.006
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	0.93	1.000	1.000	1.000	1.000	1.000	1.561

Ratio between CMF combination and CMF site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.930	0.840	0.846	0.924	0.845	0.845	0.924	0.929	0.845	0.840	0.845

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Tab. 13.54: ΔN$_{expected}$ - Combination 6.

After 10 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.930	0.048	0.015	0.033
Segment 2	0.840	0.612	0.196	0.416
Segment 3	0.846	0.049	0.016	0.033
Segment 4	0.924	0.050	0.016	0.034
Segment 5	0.845	0.054	0.017	0.037
Segment 6	0.845	0.510	0.163	0.347
Segment 7	0.924	1.590	0.509	1.081
Segment 8	0.929	0.046	0.015	0.031
Segment 9	0.845	0.062	0.020	0.042
Segment 10	0.840	0.999	0.320	0.679
Segment 11	0.845	0.060	0.019	0.041

Tab. 13.55: CMF - Combination 7.

At-site geometric characteristic

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6	2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7	2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6	2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6	1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7	3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6	1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6	2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6	1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6	1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6	4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6	1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strip*	*Auto. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.934
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	0.94	0.93	0.518	0.98	1.000	0.994	1.000	1.786
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.934
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	0.94	0.93	1.000	0.98	1.000	0.994	1.000	1.594
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	0.94	0.93	1.000	0.98	1.000	1.000	1.000	2.507
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.438
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	0.94	0.93	0.518	0.98	0.980	0.994	1.000	1.566
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.602
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	0.94	0.93	1.000	0.98	1.000	1.000	1.000	1.523
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	0.94	0.93	0.518	0.98	0.980	0.994	0.852	2.830
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	0.94	0..93	1.000	0.98	1.000	1.000	1.000	1.438

Ratio between CMF combination and CMF site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.857	0.711	0.779	0.851	0.779	0.779	0.766	0.824	0.856	0.778	0.779

Tab. 13.56: $\Delta N_{expected}$ - Combination 7.

After 10 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.857	0.098	0.031	0.067
Segment 2	0.711	1.765	0.565	1.200
Segment 3	0.779	0.100	0.032	0.068
Segment 4	0.851	0.097	0.031	0.066
Segment 5	0.779	0.112	0.036	0.076
Segment 6	0.779	1.044	0.334	0.710
Segment 7	0.766	4.913	1.572	3.341
Segment 8	0.856	0.094	0.030	0.064
Segment 9	0.778	0.125	0.040	0.085
Segment 10	0.593	1.632	0.522	1.110
Segment 11	0.779	0.123	0.039	0.084

Tab. 13.57: CMF - Combination 8.

At-site geometric characteristic

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*CMF SITE COMBINED*
Seg. 1	3.00	0.50	0.00	0.000	2.60	20.12	6	2.257
Seg. 2	3.00	0.50	170.0	0.003	4.93	0.00	7	2.513
Seg. 3	3.00	0.50	0.00	0.000	5.00	20.12	6	2.483
Seg. 4	3.00	0.50	0.00	0.000	0.03	10.06	6	1.873
Seg. 5	3.00	0.50	290.0	0.014	4.60	9.51	7	3.219
Seg. 6	3.00	0.50	0.00	0.000	4.60	0.00	6	1.847
Seg. 7	3.00	0.50	160.0	0.008	1.20	0.00	6	2.044
Seg. 8	3.00	0.50	0.00	0.000	1.20	10.01	6	1.871
Seg. 9	3.00	0.50	0.0	0.000	3.32	7.61	6	1.957
Seg. 10	3.00	0.50	80.0	0.005	3.32	10.06	6	4.768
Seg. 11	3.00	0.50	0.0	0.000	3.32	0.00	6	1.847

CMF

Homog. Seg. ID	*Lane width (m)*	*Shoulder width (m)*	*Horiz. curve Radius (m)*	*Super-elevation*	*Longitudinal slope (%)*	*Access density*	*Roadside design*	*Rumble strip*	*Auto. speed control*	*Curve signs*	*Markers & rumble strips*	*Tree adjustment*	*Guard rail*	*Friction Improvement*	*CMF SITE COMBINED*
Seg. 1	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.583
Seg. 2	1.172	1.172	2.257	1.000	1.000	1.000	1.306	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.429
Seg. 3	1.172	1.172	1.000	1.000	1.000	1.345	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.583
Seg. 4	1.172	1.172	1.000	1.000	1.000	1.115	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.313
Seg. 5	1.172	1.172	1.444	1.024	1.000	1.103	1.306	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.584
Seg. 6	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.177
Seg. 7	1.172	1.172	2.158	1.000	1.000	1.000	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.386
Seg. 8	1.172	1.172	1.000	1.000	1.000	1.114	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.311
Seg. 9	1.172	1.172	1.000	1.000	1.000	1.059	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.247
Seg. 10	1.172	1.172	4.105	1.000	1.000	1.115	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.473
Seg. 11	1.172	1.172	1.000	1.000	1.000	1.000	1.222	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.177

Ratio between CMF combination and CMF site combination (CMFcomb/CMFsite)

Seg.1	Seg.2	Seg.3	Seg.4	Seg.5	Seg.6	Seg.7	Seg.8	Seg.9	Seg.10	Seg.11
0.702	0.569	0.638	0.701	0.492	0.637	0.678	0.701	0.637	0.309	0.637

Tab. 13.58: $\Delta N_{expected}$ - Combination 8.

After 30 years-lifetime				
Homogenous segment ID	*CMF RATIO*	*ΔNexpected (Total)*	*ΔNexpected (Fatal+Injury)*	*ΔNexpected (PDO)*
Segment 1	0.702	0.615	0.197	0.418
Segment 2	0.569	0.615	0.197	0.418
Segment 3	0.638	0.627	0.201	0.426
Segment 4	0.701	0.588	0.188	0.400
Segment 5	0.492	1.066	0.341	0.725
Segment 6	0.637	6.529	2.089	4.440
Segment 7	0.678	20.301	6.496	13.805
Segment 8	0.701	0.590	0.189	0.401
Segment 9	0.637	0.785	0.251	0.534
Segment 10	0.309	9.310	2.979	6.331
Segment 11	0.637	0.770	0.246	0.524

13.7 Benefit-cost analysis

The difference between the expected average crash frequency with and without the implementation of the countermeasures (either for fatal/injury crashes or for PDO) quantifies the effect of the countermeasure. These expected variations could be converted into monetary values for each year of the countermeasure lifetime (for the short-term countermeasures the lifetime is assumed equal to 10 years, for the long-term ones, instead, is 30 years). The mean social costs set by the Report published by the Italian Ministry of Infrastructures and Transport (2010[25]) were used for the monetary estimation. The total monetary benefits from the countermeasure implementation have been estimated by using a discount rate equal to 3.5%. All the countermeasure combinations have been evaluated in order to find the most beneficial implementation on each homogenous road segment, compared to their costs.

13.7.1 Countermeasure benefits

The Annual Monetary Value (AMV) from the implementation of the Combination i can be calculated as follows. The ΔN for fatal and injury crashes and the ΔN for PDO crashes have been considered and then multiplied by the respective social costs of crashes (fatal, injury and PDO crashes) set by the previously indicated report. In Italy, the mean social cost referred to a fatal/injury crash is equal to € 309’863.00, while the one referred to PDOs is € 10’986.00. The sum of the benefits from the two estimations provides the total benefit related to the total number of crashes, $AM_{(A)}$.

The estimation of the benefit must be converted into present values related to the time of the estimation itself, by using the discount rate (3.5%). The present benefit is calculated as follows.

$$PV_{Benefits} = Total\ Annual\ Monetary\ Value\ \times CF(\,i, y) \qquad \text{(Eq. 13-28)}$$

Where CF (i, y) is the conversion factor to the current monetary value for a series of annual constant costs (see next table):

$$CF(i, y) = \frac{(1{,}0+i)^{y}-1{,}0}{i\times(1{,}0+i)^{y}} \qquad \text{(Eq. 13-29)}$$

[25] Ministero delle Infrastrutture e dei Trasporti, Dipartimento per i Trasporti, la Navigazione ed i Sistemi Informativi e Statistici, Direzione Generale per la Sicurezza Stradale (2010), *Studio di valutazione dei Costi Sociali dell’incidentalità stradale*, Report.

The present values of benefits for all the combinations are shown in the following table.

Tab. 13.59: Monetary benefits from the countermeasures and present value for each of the 8 combinations.

	Combination ID	Crash cost (Fatal+ Injury)	Crash cost (PDO)	$\Delta N_{expected}$ (Fatal+Injury)	$\Delta N expected$ (PDO)	Monetary benefits (Fatal + Injury)	Monetary benefits (PDO)	Monetary benefits (Total)	Present Value (Total)
Lifetime: 10 years	Comb.1	309,863.00 €	10,986.00 €	0.212	0.451	656,909.56 €	49,546.86 €	706,456.42 €	3,402,961.87 €
	Comb.2	309,863.00 €	10,986.00 €	0.104	0.221	322,257.52 €	24,279.06 €	346,536.58 €	1,665,087.70 €
	Comb.3	309,863.00 €	10,986.00 €	0.029	0.061	89,860.27 €	6,701.46 €	96,561.73 €	461,302.01 €
	Comb.4	309,863.00 €	10,986.00 €	0.301	0.64	932,687.63 €	70,310.40 €	1,002,998.03 €	4,829,842.24 €
	Comb.5	309,863.00 €	10,986.00 €	0.236	0.502	731,276.68 €	55,149.72 €	786,426.40 €	3,786,056.35 €
	Comb. 6	309,863.00 €	10,986.00 €	0.131	0.277	405,920.53 €	30,431.22 €	436,351.75 €	2,094,098.57 €
	Comb. 7	309,863.00 €	10,986.00 €	0.323	0.687	1,000,857.49 €	75,473.82 €	1,076,331.31 €	5,186,120.11 €
Lifetime: 30 years	Comb.8	309,863.00 €	10,986.00 €	0.536	1.140	4,982,597.04 €	375,721.20 €	5,358,318.24 €	59,272,348.01 €

13.7.2 Countermeasure costs

The following step is calculating the costs for each combination of sets of countermeasures.

The cost estimation conducted for the sets A, B and C is shown in the following table. The column “number of interventions during the lifetime” is considered as a maintenance cost or substitution cost during the 10 years of the countermeasure lifetime. The starting cost is not affected by the interest rate, while the expenses during the countermeasure lifetime are affected by it, being converted into present values. The cost of each combination must be converted into present values. These costs were divided for each year of the countermeasure lifetime. Then the mean cost has been multiplied by the aforementioned factor CF (i, y).

Tab. 13.60: Total cost of the countermeasures, sum of the countermeasure implementation and the present value over the lifetime of the countermeasures.

	Combination ID	Annual cost	Present Value - ΣCF(i,y)j*Annual costj	Total cost
Lifetime: 10 years	Combination 1	4,478.24 €	215,541.33 €	280,028.20 €
	Combination 2	2,000.00 €	96,261.61 €	136,261.61 €
	Combination 3	3,738.71 €	179,947.32 €	226,063.26 €
	Combination 4	6,478.24 €	311,802.95 €	416,289.81 €
	Combination 5	8,216.95 €	395,488.65 €	506,091.46 €
	Combination 6	5,738.71 €	276,208.94 €	362,324.88 €
	Combination 7	10,216.95 €	491,750.27 €	642,353.07 €
Lifetime: 30 years	Combination 8	19,420.00 €	6,441,413.23 €	8,771,813.23 €

	Lifetime (years)									
	1	2	3	4	5	6	7	8	9	10
CF (i,y)	1.00	1.90	2.80	3.67	4.52	5.33	6.11	6.87	7.61	8.32
	Lifetime (years)									
	11	12	13	14	15	16	17	18	19	20
CF (i,y)	9.00	9.66	10.30	10.92	11.52	12.09	12.65	13.19	13.71	14.21
	Lifetime (years)									
	21	22	23	24	25	26	27	28	29	30
CF (i,y)	14.70	15.17	15.62	16.06	16.48	16.89	17.29	17.67	18.04	18.39

The total cost of each combination is obtained by the sum of the initial costs of countermeasure implementation and the costs during the lifetime converted into present values.

13.7.3 Ranking of projects

The economic evaluation of each combination of sets of countermeasures was conducted by using the NPV (Net Present Value) and the BCR (Benefit-Cost ratio) methods. The NPV is the difference between the obtained benefit from the countermeasure implementation and the cost to implement it:

$$NPV = PV_{benefits} - PV_{costs} \quad \text{(Eq. 13-30)}$$

The NPV must be positive to justify the countermeasure implementation. The higher is the difference, the greater is the benefit of the project. Hence, the projects are listed according to their NPVs in descending order. The BCR is the ratio between the benefit and the cost of the countermeasures (or the set of countermeasures). The BCR must be greater than 1, so the best project is the one with the greatest BCR. The following table shows the calculation of the BCR and NPV for each combination.

Tab. 13.61: NPV and BCR for each combination.

	PVcosts	*PVbenefits*	*NPV*	*BCR*
Combination 1	280,028.20 €	3,402,961.87 €	3,122,933.67 €	12.15
Combination 2	136,261.61 €	1,665,087.70 €	1,528,826.29 €	12.22
Combination 3	226,063.26 €	461,302.01 €	265,238.75 €	2.04
Combination 4	416,289.81 €	4,829,842.24 €	4,413,552.43 €	11.6
Combination 5	506,091.46 €	3,786,056.35 €	3,279,964.90 €	7.48
Combination 6	362,324.88 €	2,094,098.57 €	1,731,773.69 €	5.78
Combination 7	642,353.07 €	5,186,120.11 €	4,543,767.04 €	8.07
Combination 8	8,771,813.23 €	59,272,348.10 €	50,500,534.87 €	6.76

The obtained results suggest that:
- considering the NPV, the best project should be the Combination 8, so the re-design of the road layout;
- considering the BCR, the best project should be the Combination 2 (set B), automatic speed controls;
- the BCR obtained for the combination 8 is ranked as the 6th.

It is possible to rank projects, for each homogenous segment, understanding which is the best countermeasure for each situation. This procedure suggests to the relevant road agency which are the countermeasures with the greatest impact on crash reduction. This ranking has been done according to the BCR value thanks to the incremental analysis, comparing couples of countermeasures. This procedure is more reliable than the classic ranking list obtained through the NPV (as reported in the HSM). The results of costs and benefits for each of the 7 combinations are summarized in the Table 13.62. The incremental analysis can be used only if all the BCRs are greater than 1, otherwise the costs would be greater than the benefits coming from the countermeasure implementations, so the countermeasures would be not accepted.

Tab. 13.62: NPV and BCR listed in ascending order according to the benefit-cost analysis - Combinations from 1 to 7.

	PVcosts	*PVbenefits*	*NPV*	*BCR*
Combination 2	136,261.61 €	1,665,087.70 €	1,528,826.29 €	12.22
Combination 3	226,063.26 €	461,302.01 €	265,238.75 €	2.04
Combination 1	280,028.20 €	3,402,961.87 €	3,122,933.67 €	12.15
Combination 6	362,324.88 €	2,094,098.57 €	1,731,773.69 €	5.78
Combination 4	416,289.81 €	4,829,842.24 €	4,413,552.43 €	11.6
Combination 5	506,091.46 €	3,786,056.35 €	3,279,964.90 €	7.48
Combination 7	642,353.07 €	5,186,120.11 €	4,543,767.04 €	8.07

All the projects have BCR greater than 1. The combination 8 was neglected because of environmental restrictions which impede relocating the road layout in the surrounding area. Hence, the incremental analysis for the chosen combinations could be done. The incremental analysis (another analysis suggested by the HSM[1]) compares couples of combinations, basing it on the difference between the benefit of the second cheapest project and the benefit of the cheapest project. The same difference is applied to the costs. The ratio between these two differences ($\Delta_{benefits}$ and Δ_{costs}) gives the incremental BCR. If the incremental BCR is greater than 1, the most expensive project among the considered two, wins. Otherwise the cheapest one wins. The winning project is than compared to the third cheapest project and the same procedure is applied. The project which results as the "winner" after all comparisons is the best at all in terms of benefits-costs. By removing the losing projects, projects are ranked in terms of convenience. The last one is the less convenient based on the incremental analysis.

In this case, the "winning" project is the combination 7, related to the implementation of the sets A, B and C at the same time. The obtained ranking list is the following.

Tab. 13.63: Combination hierarchy (based on the incremental BCR analysis).

Ranking	*Combination ID*	*PVcosts*	*PVbenefits*	*NPV*	*BCR*
1	Combination 7	642,353.07 €	5,186,120.11 €	4,543,767.04 €	8.07
2	Combination 4	416,289.81 €	4,829,842.24 €	4,413,552.43 €	11.6
3	Combination 5	506,091.46 €	3,786,056.35 €	3,279,964.90 €	7.48
4	Combination 1	280,028.20 €	3,402,961.87 €	3,122,933.67 €	12.15
5	Combination 6	362,324.88 €	2,094,098.57 €	1,731,773.69 €	5.78
6	Combination 2	136,261.61 €	1,665,087.70 €	1,528,826.29 €	12.22
7	Combination 3	226,063.26 €	461,302.01 €	265,238.75 €	2.04

Combination 7 seems to be effective to mitigate or eliminate some critical situations observed during the diagnosis phase, even if apparently is not related to the crash occurrence, like friction loss and problems in sight distance. The automatic speed control and the new posted speed, 60 Km/h, should avoid the friction loss for wet road conditions providing a higher safety level to the analysed road segment. The countermeasures suggested by the Combination 7 have been implemented not only where crashes have been detected, but also in some road parts where no crashes have occurred. This strategy is justified by the idea of checking the effectiveness and reliability of the chosen countermeasures (traffic signs road markers, electronic speed control). Other countermeasures were implemented on the dangerous curves to increase their visibility and alert drivers, like curve warning signs, LED markers, rumble strips. In the most dangerous part of the analysed road segment, barriers were implemented to protect the dry-stone walls and other structures; Anti-skid road pavement at the Curve 4, as suggested by the Combination 4.

The choice of countermeasures and their extent depends on the available budget and the priorities set by the road agencies and public administrations. However, the best solution is giving all the possible alternatives to the road manager/owner and let them decide with the complete awareness of benefits and costs, for each alternative.

13.8 Focus on countermeasures for accesses and intersections

In the example of application shown, the focus was on a road segment without any significant intersection with similar roads on it. To fill this gap, in this paragraph, some examples of countermeasures in case of presence of intersections are provided. Clearly, intersection-related countermeasures are particularly important when most crashes occur at intersections and/or accesses. In the example shown here (Figure 13.28, road section in the Metropolitan City of Bari: SP231), the main causes of these crashes are the high speeds of vehicles, allowed by road characteristics, also while crossing the minor intersections and the accesses. There are no speed cameras and long tangents induce car drivers to overtake heavy vehicles even in case of lack of visibility. Moreover, the elevation profile contributes to make unclear the road perception by drivers. The collision diagram and the horizontal alignment of a segment of the analysed road are shown below.

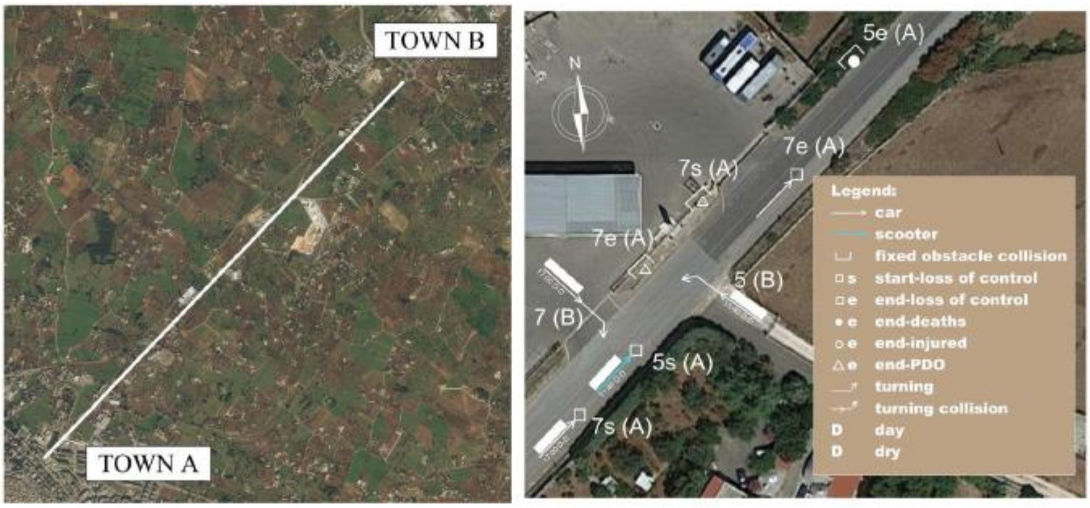

Fig. 13.28: Horizontal alignment and collision diagram of the example road segment characterized by many accesses and minor intersections (Colonna et al., 2018[5]).

Design interventions on this part of the road are strongly advised to avoid the occurrences of crashes. The most recommended one is the use of secondary roads or, at least short frontage roads, to collect the minor accesses before inserting their vehicles' flow into the primary road one. When this approach is not possible due to environmental and physical boundaries, the most feasible and affordable countermeasures (short-term) can be speed cameras, so forcing drivers to slow down, and the implementation of road danger signs near driveways.

If there is a sufficient available space to modify the road geometry, and there are no other valid restrictions to this choice, designing a roundabout is the most proper solution to solve the minor intersection issues. All the driveways would converge into the roundabout reducing the dangerous point of conflicts. The aforementioned suggested solutions and countermeasures thought to sort out the highlighted issues of accesses, and minor intersections, are shown in figures below.

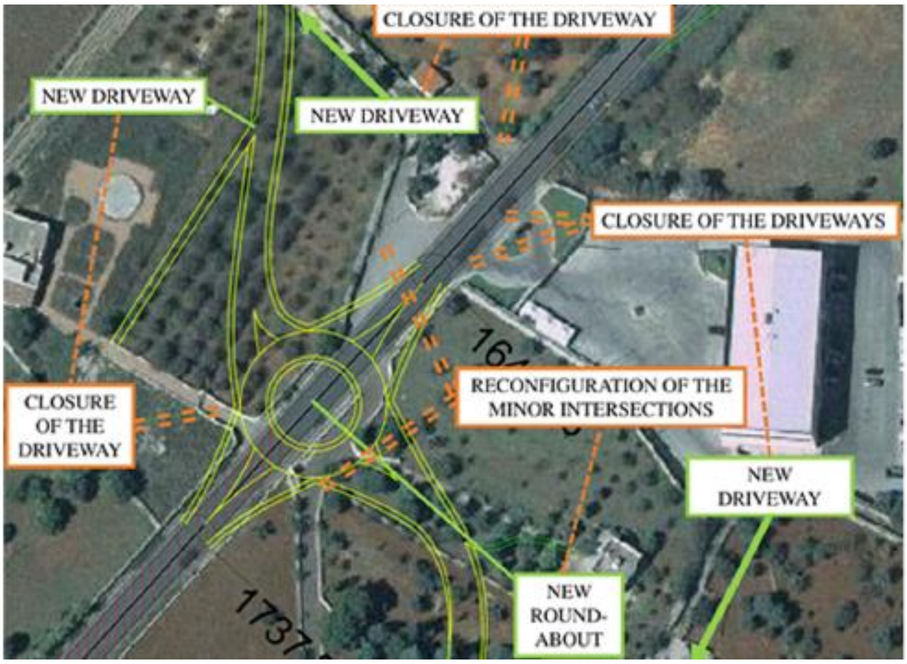

Fig. 13.29: Example of roundabout for enhancing minor intersection geometry (Colonna et al., 2018[5]).

Fig. 13.30: Example of secondary roads for the collection of flows from minor intersections (Colonna et al., 2018[5]).

13.9 References

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14. Example of design application: urban roads

14.1 Introduction

In this chapter, the proposed protocol for safety interventions on existing roads is applied to the urban environment. The applied procedure is similar to that explained in the previous chapter dedicated to safety interventions on existing rural roads.

However, the design protocol had been adapted for taking into account the influence of some specific urban issues.

Nevertheless, the steps composing the design protocol are the same indicated in the previous chapter.

For illustrative purposes, a case study example was used to explain the design procedure in the urban environment. It refers to some segments and intersections located in the city of Bari, Italy.

14.2 General background for the urban case study

The part of the road network investigated is centered around the main 4-legged signalized intersection between the roads: “Viale Domenico Cotugno”, “Viale Papa Giovanni XXIII”, “Viale Orazio Flacco” and “Viale Papa Pio XII”. The road segments and the respective intersections investigated in this chapter are located in the city of Bari (Italy), see next figure.

This intersection was selected for the example of application because it shows a high potential for safety improvements, as shown in the following. In fact, as already shown in the rural case, the average expected crash frequency is compared with the results from some relevant urban Safety Performance Functions (for 4-legged or signalized urban intersections).

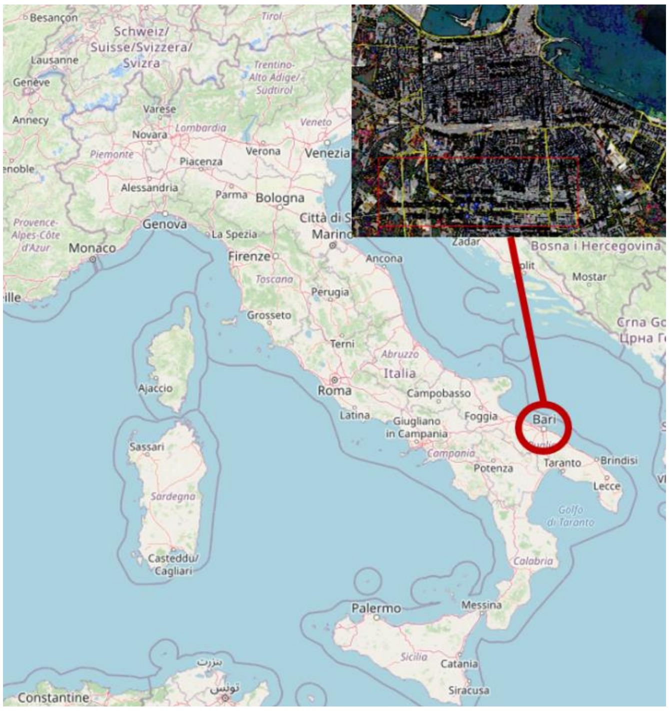

Fig. 14.1: General overview – road segments and intersections in the city of Bari (photo source OpenStreetMap/Google Earth).

This comparison highlights the excess in crash frequency of the selected site with respect to average estimates from the SPF (Figure 14.2). Hence, it is an optimal candidate for this example of application. Segments and intersections close to this specific intersection are studied as well, in order to consider a part of road network besides a single segment or intersection, to be closer to real safety enhancement projects on the existing network.

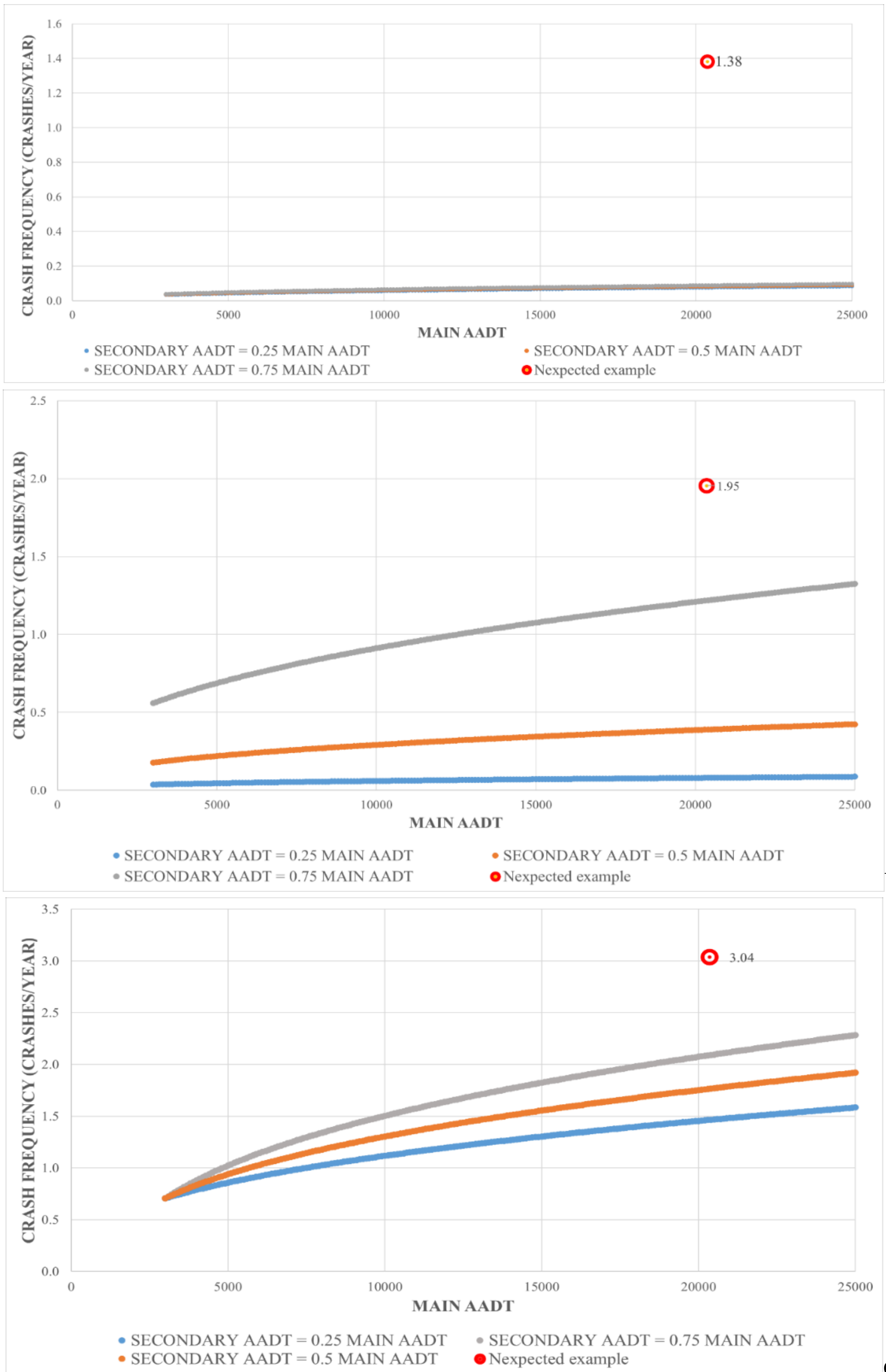

Fig. 14.2: Comparison of the $N_{expected}$ calculated for the example of application with some SPFs developed by a) Gomes et al. (2012)[1] for 4-legged intersections, b) Intini et al. (2020)[2] for signalized intersections, c) Intini et al. (2020)[2] for 4-legged intersections. See chapter 14.10 for more information about those SPFs and reference studies.

[1] Gomes S. V., Geedipally S. R., Lord D. (2012), "Estimating the safety performance of urban intersections in Lisbon, Portugal", *Safety science*, 50(9), 1732-1739.

[2] Intini P., Berloco N., Cavalluzzi G., Colonna, P., Lord D., Ranieri V. (2020), "The variability of urban safety performance functions for different road elements: an italian case study", Under review.

The specific road segments investigated (shown in blue in figure 14.3) are:

- Viale Orazio Flacco (segment between Via Storelli and Via Papa Giovanni XXIII)
- Viale Papa Giovanni XXIII (from the intersection with Via Orazio Flacco, Via Cotugno and Viale Papa Pio XII to the intersection with Via Lioce, both of them analyzed)
- Via Niceforo (including the intersection with Via Papa Giovanni XXIII)
- Via Poli (including the intersection with Via Papa Giovanni XXIII).

14.2.1 General view of the intersection I

The first investigated intersection is shown in the figure below.

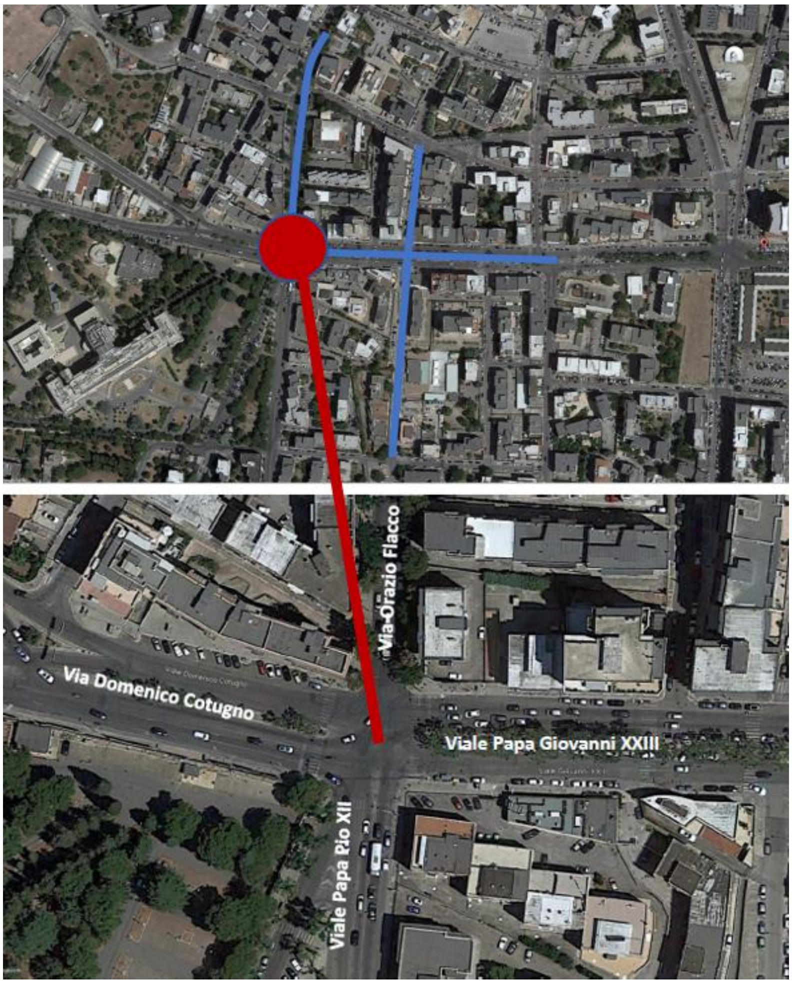

Fig. 14.3: Overview of the investigated segments (above in blue) and detail of the highlighted high-crashed intersection (below): Viale Papa Giovanni XXIII, Viale Papa Pio XII, Via Domenico Cotugno and Via Orazio Flacco (photo source Google Earth).

The intersecting road segments in the main node will be studied as well. In fact, in order to correctly study what happens in the intersection, a deep analysis of the intersecting road segments is necessary.

The first step of the study is addressing to each road segment its function in the territory, either considering the road segment alone or considering it in the general road network. The intersection in exam has an important role in the city context. Such intersection is a 4-legged intersection where Viale Papa Giovanni XXIII, Viale Papa Pio XII, Via Domenico Cotugno and Via Orazio Flacco converge. Via Cotugno (East-West direction) is essentially the prosecution of Via Generale Nicola Bellomo, which is the road linked to the exit 10A (Bari Picone) of the SS16 directed towards the Picone, Carrassi and Poggiofranco neighbourhoods.

This road goes from the intersection with Via delle Murge to the analysed intersection. Via Cotugno receives too the traffic flow of vehicles coming from Bitritto (SP 236), Loseto and Ceglie del Campo (SP 183), heading to Bari. Viale Orazio Flacco (South-North direction) is one of the most used roads in the main Hospital area. Its extension goes from the analysed intersection to the roundabout in front of the main Hospital, Piazza Giulio

Cesare. This is congested by traffic flow both entering into the city (morning hours) and going out of it (night hours). The high frequency of emergency vehicles passing in this road to reach the hospital itself is not negligible. Viale Papa Pio XII (South-North direction) connects the Poggiofranco neighbourhood to the city centre.

It goes from the intersection with Via J.F. Kennedy to the analysed one. There is the Oncological Hospital Giovanni Paolo II in this street. Viale Papa Giovanni XXIII (East - West direction) starts from the intersection of Via Solarino with Via Cotugno around the analysed intersection. It is a part of a main urban arterial, ending where Parco 2 Giugno is located. This street is the boundary line between the neighbourhoods: Poggiofranco/Picone, Carassi/San Pasquale.

14.2.2 General view of the intersection II

The second investigated intersection is shown in the figure 14.4.

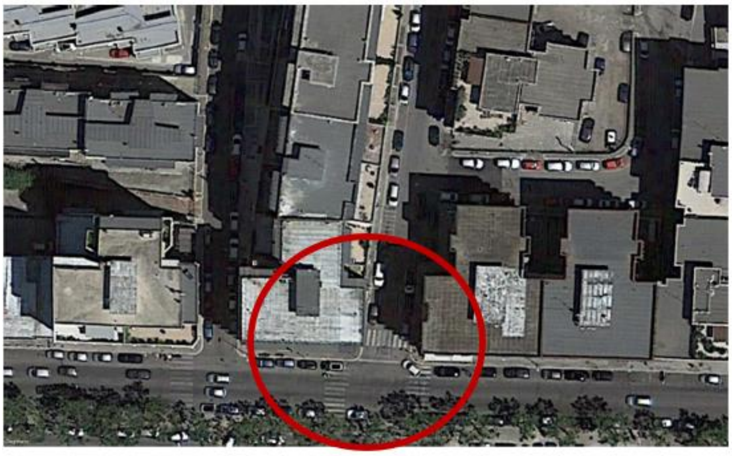

Fig. 14.4: Intersection between Viale Papa Giovanni XXIII and Via Giuseppe Saverio Poli (photo source Google Earth).

This is a 3-legged intersection where Viale Papa Giovanni XXIII and Via Giuseppe Saverio Poli converge. Via Giuseppe Saverio Poli (South-North direction) allows the connection with the residential area and with local roads connected to private properties. It connects Via Papa Giovanni XXIII to the hospital Santa Maria.

14.2.3 General view of the intersection III

The third investigated intersection is shown in the figure 14.5.

Fig. 14.5: Intersection between Viale Papa Giovanni XXIII and Via Niceforo (photo source Google Earth).

Such intersection is a 3-legged intersection where Viale Papa Giovanni XXIII and Via Niceforo converge. Via Niceforo (South-North direction) allows the connection with the residential area and the local roads connected to private properties. It connects Viale Papa Giovanni XXIII with Viale J. F. Kennedy. These two roads are similar in terms of functional and geometric characteristics.

14.2.4 General view of the intersection IV

This is a 4-legged intersection where Viale Papa Giovanni XXIII, Via Saverio Lioce, Via Giovene converge.
Via Saverio Lioce (South-North direction) is an important connection between Viale Papa Giovanni XXIII and Via J.F. Kennedy. It allows the connection with the residential area.
Via Giovene (South-North direction) allows the connection with the residential area and local roads connected to private properties.

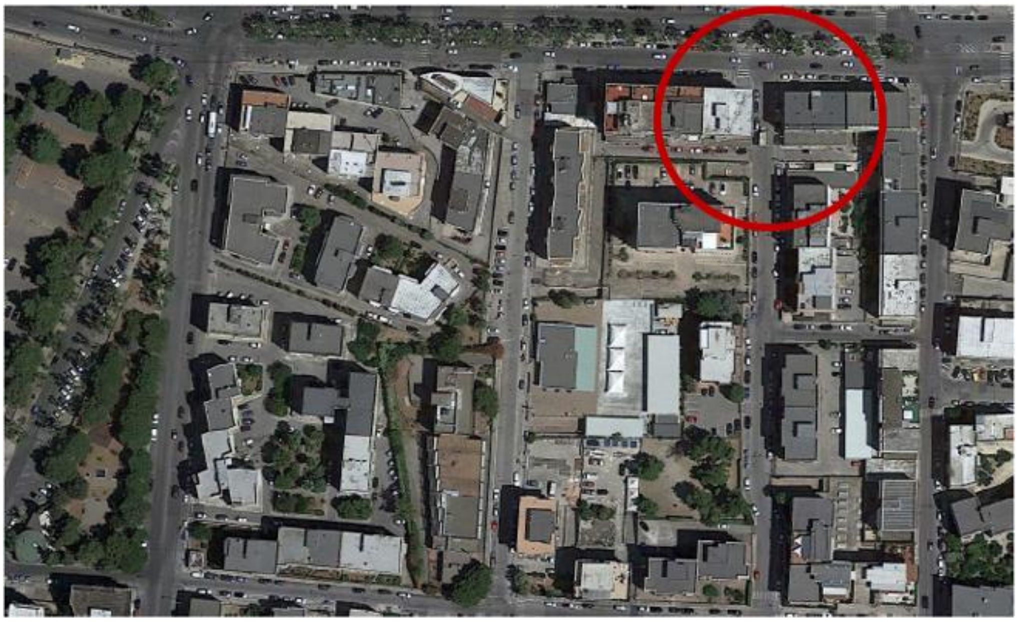

Fig. 14.6: Intersection between Viale Papa Giovanni XXIII, Via Saverio Lioce, Via Giovene (photo source Google Earth).

14.3 Category and function of the road segments

14.3.1 Function of the road segments

The Italian road standards D.M. 2001[3] states the need for classifying the role of the single road segment in the network, according to the following factors:

- type of provided movement. It could be a transit movement, a distribution movement, a penetration movement and an access movement;
- importance of the movement, related to the average distances travelled by the vehicles;
- function in the area where the road is placed;
- traffic components and categories of traffic components (light vehicles, heavy vehicles, motorcycles, pedestrians, etc.).

According to the aforementioned elements, four road network levels exist, as shown in Table 14.1.

[3] Ministerial Decree n. 6792 of 5 November 2001, *Functional and Geometric Road Standards -Norme Funzionali e Geometriche delle Strade*.

Tab. 14.1: 4 Road network levels according to the D.M. 6792/2001[3]. Primary network (a), main network (b), secondary network (c), local network (d) and the respective road types for urban and rural area.

Network	*Road Type According to the Code*	
	rural area	*urban area*
a- primary network	rural freeways	urban freeways
(transit function)	main rural roads	transit urban roads
b - main network	main rural roads	transit urban roads
(distribution function)		
c - secondary network	secondary rural roads	neighbourhood roads
(penetration function)		
d - local network	local rural roads	local urban roads
(access function)		

The following figure (14.7) provides a graphic representation of the table 14.1.

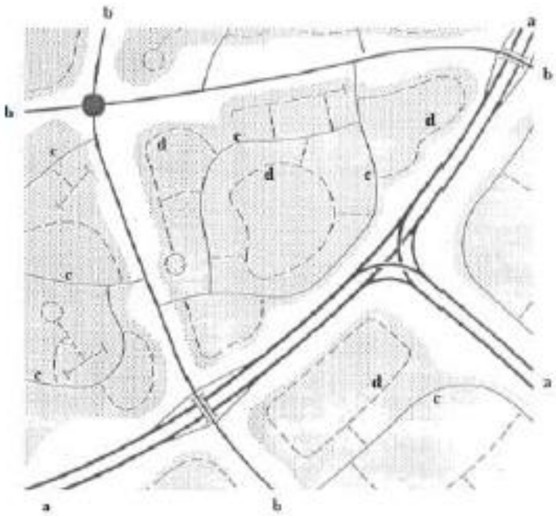

Fig. 14.7: Road network levels according to the D.M. 6792/2001[3].

According to the D.M. 6792/2001[3], the road network level for the considered case can be identified. The investigated roads are signed as "b"-level, main network and distribution function. In fact, the main roads under analysis are transit roads on which local or neighbourhood roads intersect. The users of this network make movements between different neighbourhoods or heading to rural roads.

The D.M. 6792/2001[3] defines the functions of each single arch belonging to the network. It establishes a main function and a secondary function, which should correspond to the main function of the adjacent level in order to provide an organic and homogenous service. This is shown in Table 14.2.

Tab. 14.2: Primary, main, secondary and local road type related to their functions: transit, distribution, penetration and access. The black dots stand for the main function and the white ones for the secondary function[3].

ROAD TYPE / *FUNCTION*	*PRIMARY*	*MAIN*	*SECONDARY*	*LOCAL*
transit	•	○		
distribution	○	•	○	
penetration		○	•	○
access			○	•

• Main function
° Secondary function = Main function of the adjacent class

Thanks to the D.M. 6792/2001[3] indications, the considered roads are classified as follows:

- via Cotugno and Viale Papa Giovanni XXIII:
  - (I function) b-level main network/distribution function as main function because they link different neighbourhoods (Picone, Carassi, Poggiofranco and San Pasquale);
  - (II function, adjacent level) c-level secondary level/penetration function because they penetrate in the residential area through intersections with neighbourhood roads and local roads.
- Viale Orazio Flacco and Viale Papa Pio XII:
  - (I function) c-level, secondary network/penetration function, in fact they penetrate in the residential area, in residential neighbourhood like Picone, Poggiofranco and Carrassi;
  - (II function, adjacent level) b- level, principal network/distribution function.
- Via Giuseppe Saverio Poli and Via Niceforo:
  - (I function) c-level, secondary network/penetration function, in fact they penetrate in the residential area;
  - (II function, adjacent level) d-level, local network/access function because they allow to reach the private properties.
- Via Lioce:
  - (I function) c-level , secondary network/penetration function, because it penetrates in residential areas;
  - (II function, adjacent level) b-level principal network/distribution function.

14.3.2 Category of the road segments

According to what is prescribed by both the Codice della Strada (Italian Road Regulations) and the D.M. 6792/2001[3], roads are classified as based on their design, technical and functional characteristics in 8 different categories, 4 for the urban context (A_U, D, E and F_U) and 4 for the rural context (A, B, C and F).

Tab. 14.3: Categories of roads, territorial context and their speed limits (lower design speed, upper design speed and posted speed) from D.M. 6792/2001[3].

						Design speed range	
Road Type According to the Code	*Id*	*Context*		*Posted Speed*	*Lanes for direction of travel*	*Lower design speed (km/h)*	*Upper design speed (km/h)*
1	2	3		4	5	6	7
Freeway	A	Rural	main road	130	2 or more	90	140
			possible service road	90	1 or more	40	100
		Urban	main road	130	2 or more	80	140
			possible service road	50	1 or more	40	60
Main Rural	B	Rural	main road	110	2 or more	70	120
			possible service road	90	1 or more	40	100
Secondary Rural	C	Rural	C1	90	1	60	100
			C2	90	1	60	100
Transit Urban	D	Urban	main road	70	2 or more	50	80
			possible service road	50	1 or more	25	60
Neighbourhood Urban	E	Urban		50	1 or more	40	60
Local	F	Rural	F1	90	1	40	100
			F2	90	1	40	100
		Urban		50	1 or more	25	60

Tab. 14.4: Categories of roads, territorial context, regulations for parking, public transport, pedestrian traffic and accesses[3].

Road Type According to the Code	*Id*	*Context*		*Posted Speed*	*Number of Lanes for direction of travel*	*Lower design speed (km/h)*	*Upper design speed (km/h)*
1	2	3		18	19	20	21
Freeway	A	Rural	main road	Allowed in separated spaces with concentrated entries and exits	Not allowed the stop	Not allowed	Not allowed
			possible service road	Allowed in ad-hoc spaces (parking lots)	Stops arranged in special areas on the carriageway side	On shoulder	Allowed
		Urban	main road	Allowed in separated spaces with concentrated entries and exits	Not allowed the stop	Not allowed	Not allowed
			possible service road	Allowed in ad-hoc spaces (parking lots)	Stop areas or possible reserved lane	On protected sidewalks	Allowed
Main Rural	B	Rural	main road	Allowed in separated spaces with concentrated entries and exits	Allowed in separated spaces with special entries and exits	Not allowed	Not allowed
			possible service road	Allowed in ad-hoc spaces (parking lots)	Stops arranged in special areas on the carriageway side	On shoulder	Allowed
Secondary Rural	C	Rural	C1	Allowed in lay by	Stops arranged in special areas on the carriageway side	On shoulder	Allowed
			C2				
Transit Urban	D	Urban	main road	Allowed in separated spaces with concentrated entries and exits	Reserved lane and/or special stop areas	On protected sidewalks	Not allowed
			possible service road	Allowed in ad-hoc spaces (parking lots)	Stop areas	On sidewalks	Allowed
Neighbourhood Urban	E	Urban		Allowed in ad-hoc spaces (parking lots)	Stop areas or possible reserved lane	On sidewalks	Allowed
Local	F	Rural	F1	Allowed in lay by	Stops arranged in special areas on the carriageway side	On shoulder	Allowed
			F2				
		Urban		Allowed in ad-hoc spaces (parking lots)	Stop areas	On sidewalks	Allowed

From a functional point of view, Via Cotugno and Viale Papa Giovanni XXIII are defined as a D category,

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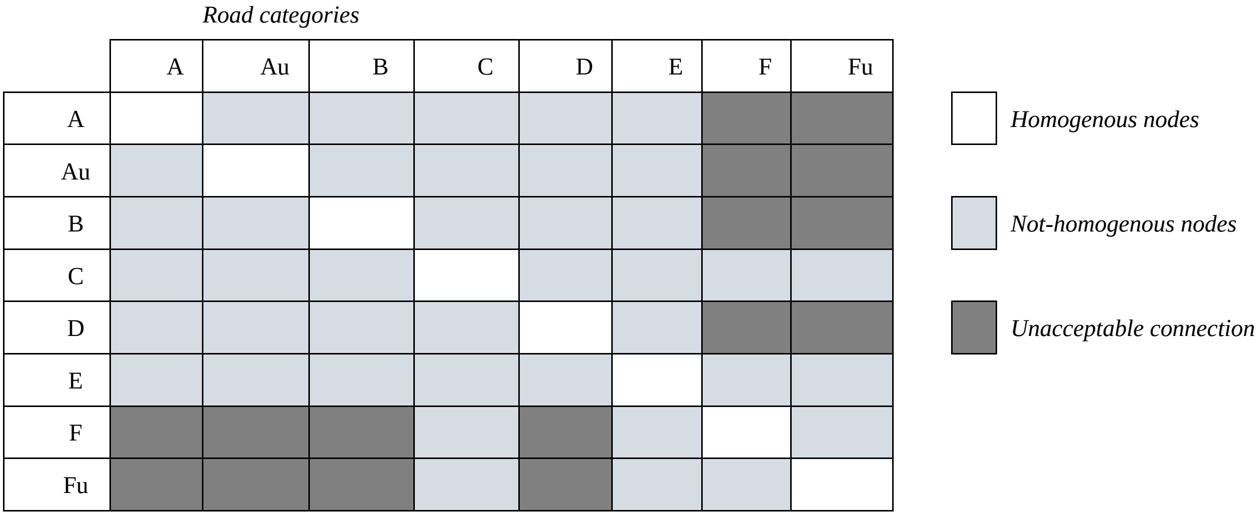
even they correspond to an E category according to the geometric characteristics. In order to be effectively a D road, all accesses should not be allowed, as well as the parking areas on the road. Moreover, the pedestrians might be protected by barriers. Viale Papa Pio XII e Via Orazio Flacco are E category roads.

Via Cotugno (in the final part, the one analysed) and Viale Papa Giovanni XXIII have separated carriageways by traffic islands, and two lanes for each direction. Viale Orazio Flacco has one carriageway with two lanes, one for each direction. Viale Papa Pio XII has one carriageway with four lanes, two for each direction. Via Saverio Lioce, instead is an E Category road, with one carriageway with two lanes, one lane for each direction (so it is also geometrically an E category road). Via Giuseppe Saverio Poli and Via Niceforo are two F category roads with one carriageway and one lane for each direction according to their geometric characteristics.

14.3.3 Analysis of the interconnection nodes

A symmetric matrix with rows and columns filled with the categories of roads is the best way to represent the interconnection nodes. The matrix exhibits all the possible combinations of nodes between all the categories of roads. Homogenous nodes (node between two roads of the same category) and not-homogenous nodes (node between two roads of different categories). Not-homogenous nodes could be unacceptable, so it is not possible to design an intersection neither for functional reasons nor for safety reasons, or acceptable.

Tab. 14.5: Acceptable and not acceptable node interconnections. Grey squares stand for unacceptable connections, white ones for homogenous nodes and the light blue squares are the not homogenous but acceptable connections; from D.M. 19/04/2006 (Italian regulations about intersections).



The intersection "Via Cotugno - Viale Papa Giovanni XXIII - Viale Papa Pio XII - Viale Orazio Flacco" is between two E category roads and two D category roads, so this node is not-homogenous according to the D.M. 19/04/2006 (Italian regulations about intersections).

But if Viale Papa Giovanni XXIII is classified as a road of category E, instead of D for the aforementioned geometric reasons, the node becomes homogenous. Considering Viale Papa Giovanni XXIII as a E category road henceforth, the intersections between Viale Papa Giovanni XXIII and Via Poli, as well as the intersection between Viale Papa Giovanni XXIII and Via Niceforo are acceptable are not-homogenous but acceptable (if Viale Papa Giovanni was a D category road the node would have been not acceptable). The intersection between Viale Papa Giovanni XXIII, Via Giovene and Via Lioce is characterized by a homogenous node because all the roads are E category roads.

14.4 Geometric reconstruction

The geometric reconstruction of the road segments is a fundamental step to recreate the road network model as close as possible to the actual situation of segments and intersections.

14.4.1 Geometric reconstruction of the road horizontal alignment road centerline

In a preliminary stage, information about the geometry of the road elements were obtained by means of digital maps and aerial photos. Map layers and the aerial photos were overlapped in a CAD/GIS environment. The horizontal alignment of the curbs has been drawn in red as shown in figure 14.8.

Fig. 14.8: Reconstruction of the curbs, in red. (photo source Google Earth).

All the road segments look like to be tangents, while a part of Viale Orazio Flacco can be considered as a curve. After the curbs, the road centreline has been reconstructed. Thus, each road segment has been defined by curbs and road centrelines. Hence, the Viale Orazio Flacco reconstruction was as follows: one tangent from the analysed intersection to the beginning of the first curve, the first curve, then the second curve (with different radius and length) and finally, the last tangent until the Hospital square.

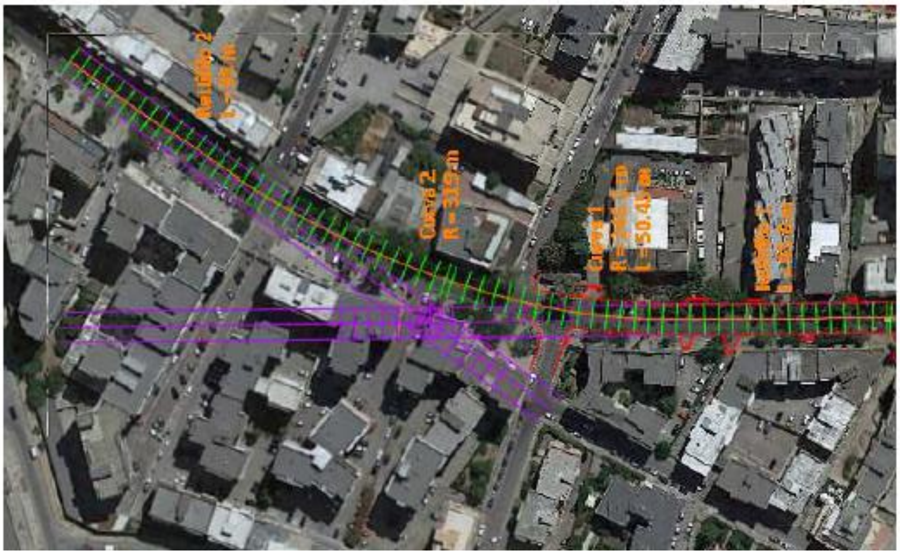


Fig. 14.9: Viale Orazio Flacco reconstruction (photo source Google Earth).

Viale Papa Giovanni XXIII exhibits two equal carriageways on the two sides of the median, so the cross section was drawn on the median (whose width is 3.00 m), simplifying the reconstruction procedure.

Fig. 14.10: Viale Papa Giovanni XXIII reconstruction (photo source Google Earth).

The output of the whole reconstruction procedure of the curbs and road centreline is shown in figure 14.11.

Fig. 14.11: Whole reconstruction of the road curbs and centrelines of the analysed area (photo source Google Earth).

Starting from the graphical reconstruction, it was possible to list all the parameters of the geometric elements composing the road segment layout. The parameters are listed in the table below.

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Tab. 14.6: Summary of the road geometric characteristics.

	Road Element		
Road Name	*Tangent - Length L [m]*	*Curve - Length S and Radius R [m]*	*Total Length [m]*
Viale Papa Giovanni XXIII	L1 = 276.00		L = 276.00
Viale Orazio Flacco	L1 = 95.60	S1 = 50.45 R1 = 246.2	L = 146.05
Via Saverio Poli	L1 = 96.60		L =96.60
Via Niceforo	L1 = 185.00		L= 185.00

14.4.2 Reconstruction of the horizontal and vertical traffic signs

In order to focus on the context in which the geometric reconstruction is performed, traffic signs, both horizontal and vertical should be taken into account. This information could provide a first indication of the road safety conditions, because the traffic signs affect the user risk perception risk. Billboards and light posts were considered in the category of traffic signs.

The detected horizontal traffic signs are the following.

- Lane edge lines
- On-street parking lines
- Lines delimiting the junk drawer areas
- Specialized lanes at the intersection ruled by traffic lights
- Zebra patterns for areas where the parking is not allowed
- Crosswalks
- Stop and give-way lines
- Bus stops and edge lines of bus lanes

The vertical traffic signs are the following.

- All vertical road signs
- Bus stop signs
- Traffic lights
- Cameras for speed/traffic lights violations detection.

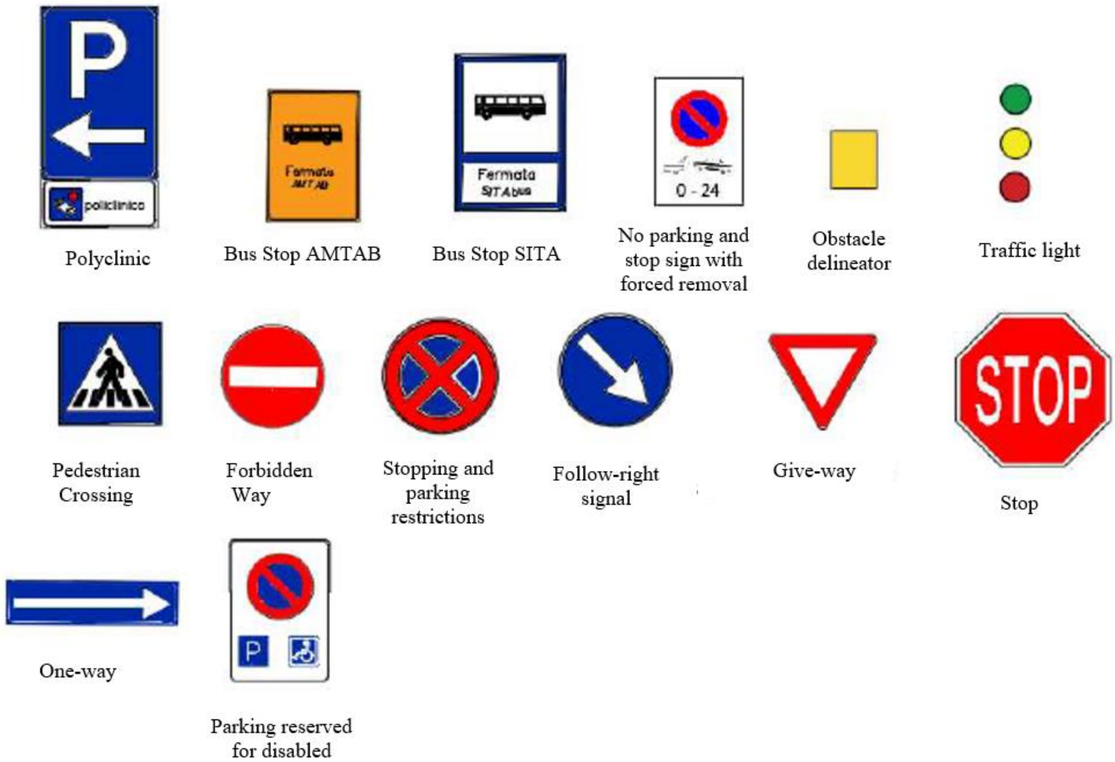

Fig. 14.12: Examples of detected vertical traffic signs.

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14.4.3 Standard cross section of the analysed segments and intersections and comparison with the cross section provided by the D.M. 6792/2001 – Norme funzionali e geometriche per la costruzione delle strade

This section shows the procedures useful to identify the road type, by comparing the actual characteristics of the road segments and the geometrical characteristics that each road category should have according to the Italian Standards D.M. 6792/2001[3] – *Norme funzionali e geometriche per la costruzione delle strade* (listed in the previous Table 14.1).

The first step is to determine the standard cross section of each road segment included in this case study and the cross section of the segments approaching to the intersections (i.e., from 2 to 5 m before the intersection itself). The road segment standard cross section was easy drawn thanks to the evidences obtained by reconstructing the road design elements. Even if the shoulders were absent in most of the analysed cases, they were added in the standard cross section in order to compare it with is the cross section provided by the D.M. 6792/2001[3]. As previously mentioned, the dimensions of the lanes were defined thanks to digital tools but also in site, through ad-hoc equipment, in order to have reliable measurements. Since the measures of the road cross section may vary, these variations have been considered in the standard cross section to provide a synthetic view of its characteristics. All the cross sections drawn are shown in the figures below:

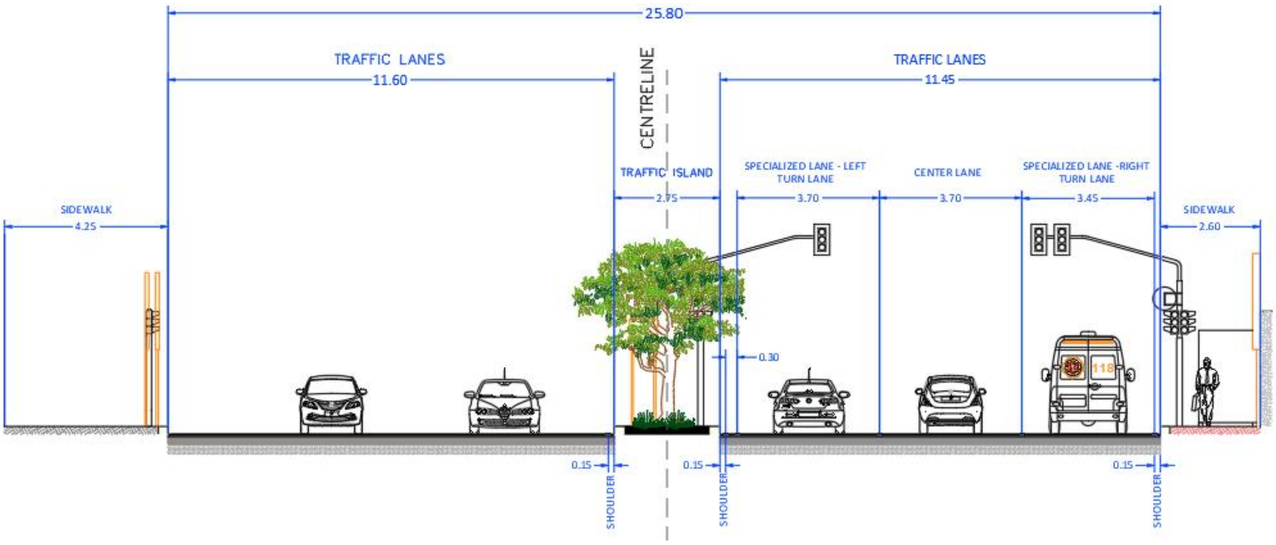

Fig. 14.13: Cross section, intersection I - Via Cotugno.

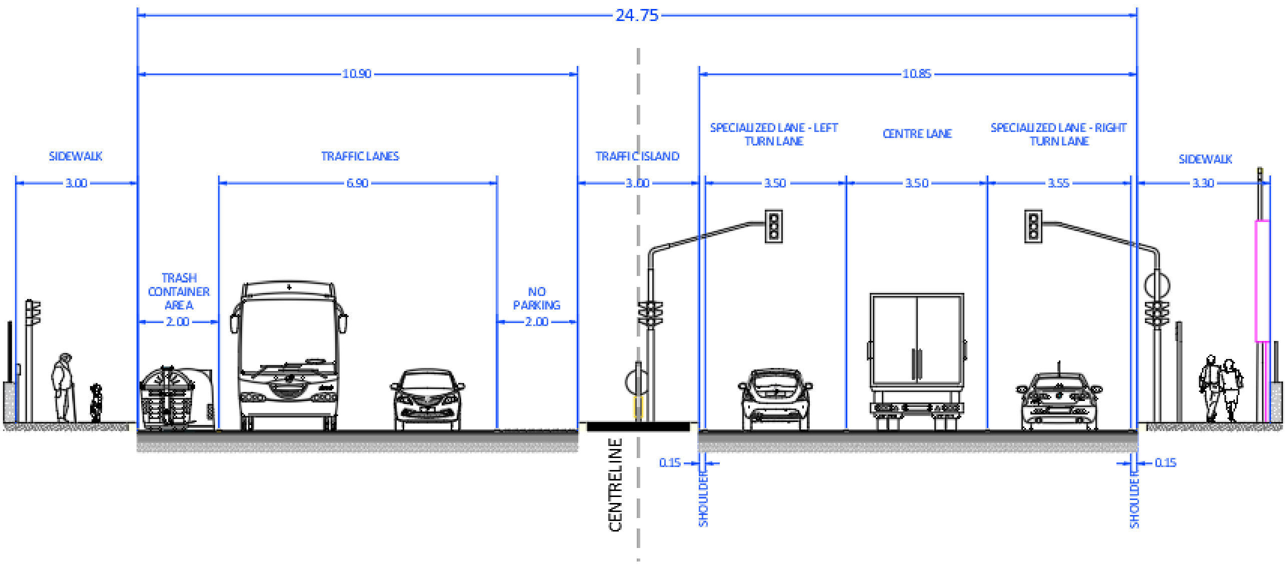

Fig. 14.14: Cross section, intersection I - Viale Papa Giovanni XXIII.

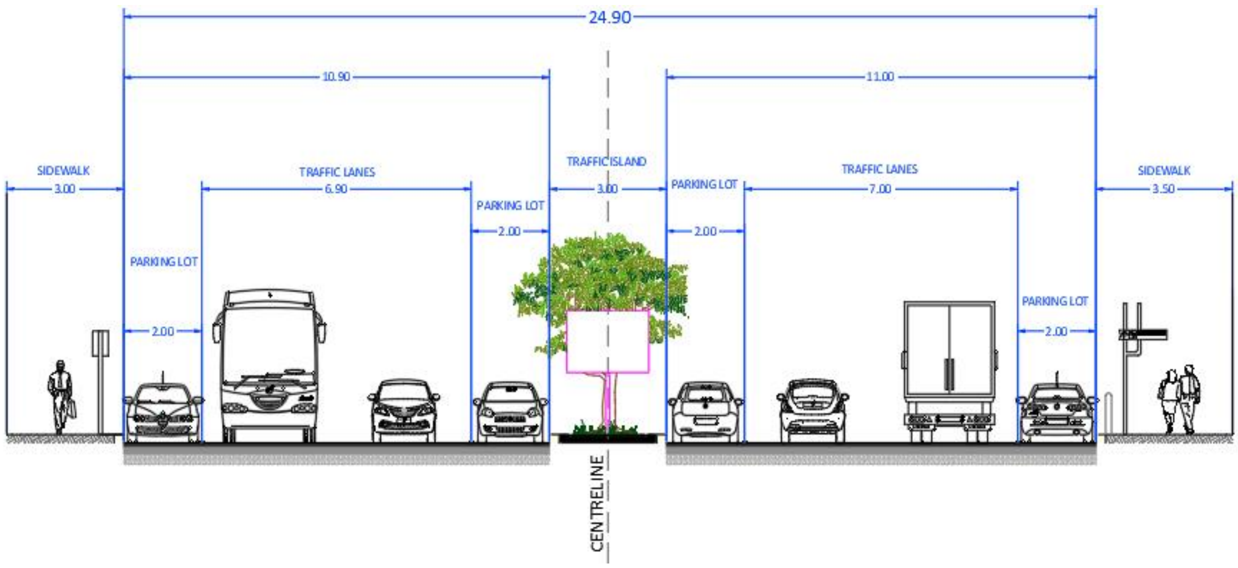

Fig. 14.15: Cross section, section F-F'; road segment of Viale Papa Giovanni XXIII.

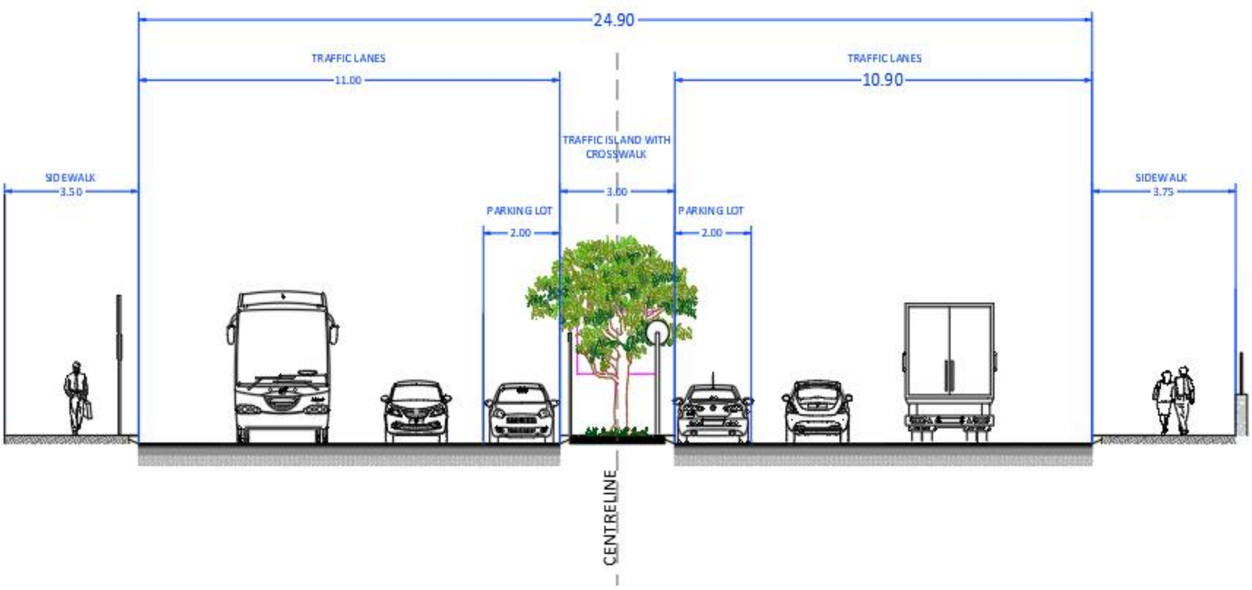

Fig. 14.16: Cross section, section G-G', intersection II and III - Viale Papa Giovanni XXIII.

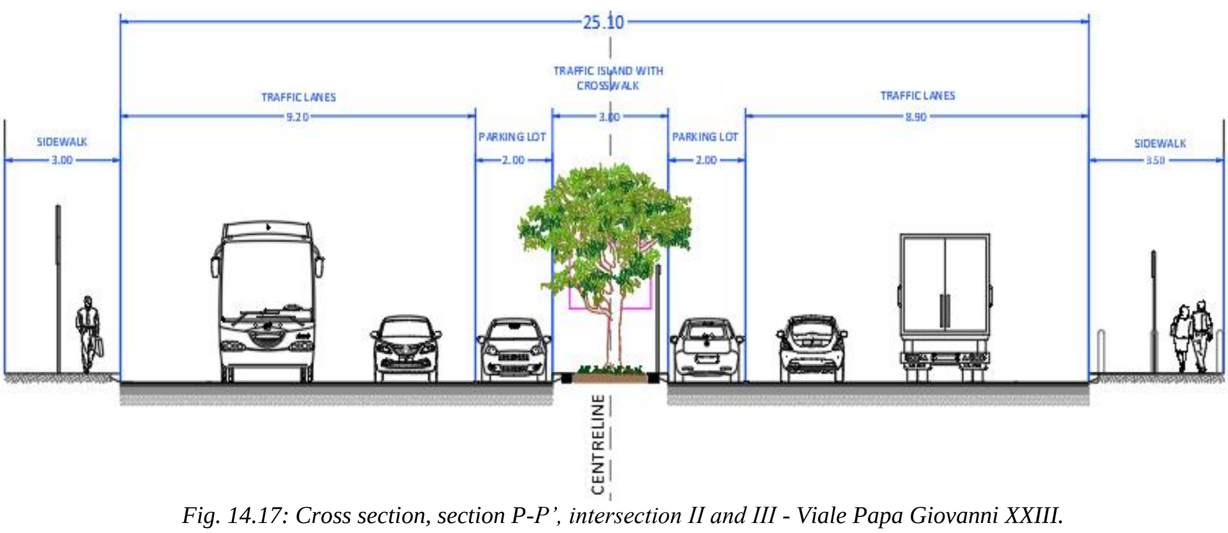

Fig. 14.17: Cross section, section P-P', intersection II and III - Viale Papa Giovanni XXIII.

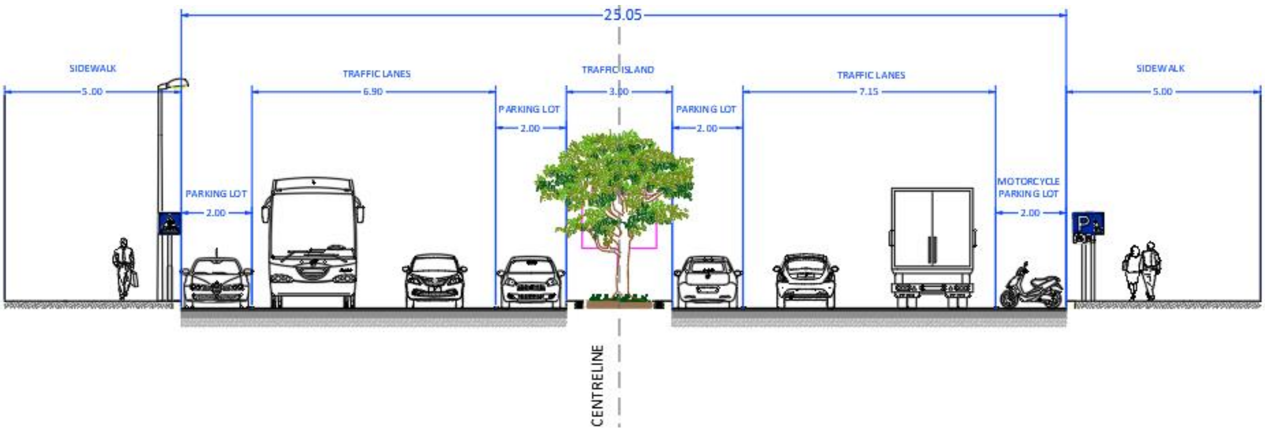

Fig. 14.18: Cross section, section Q-Q'; road segment of Viale Papa Giovanni XXIII.

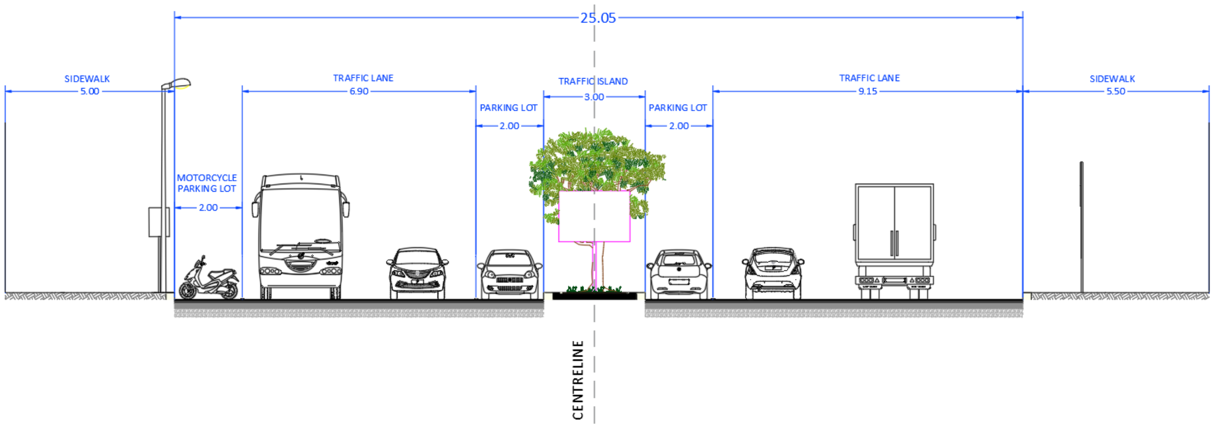

Fig. 14.19: Cross section, section R - R'; road segment of Viale Papa Giovanni XXIII.

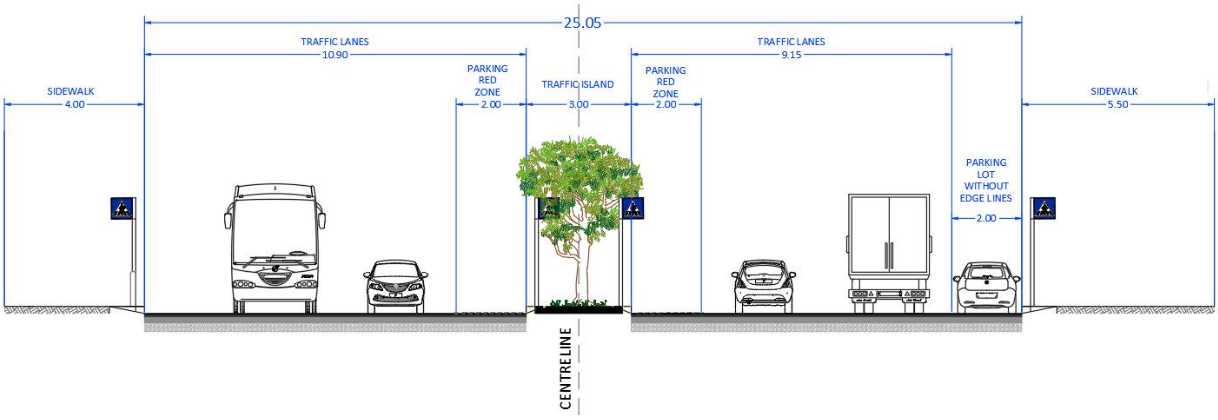

Fig. 14.20: Cross section, section S - S'; road segment of Viale Papa Giovanni XXIII.

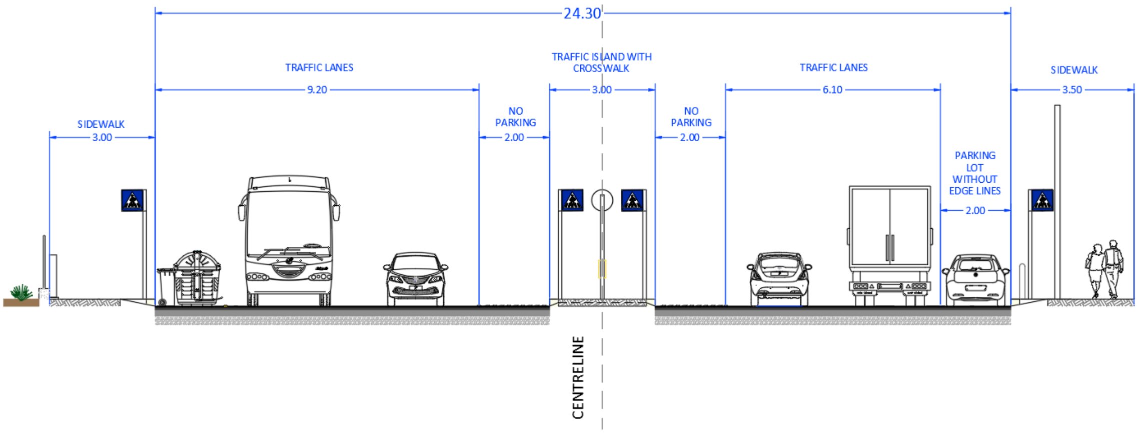

Fig. 14.21: Cross section, section U-U', intersection IV - Viale Papa Giovanni XXIII.

Based on the previous classification of the road segment Viale Papa Giovanni XXIII, it has a main function: b - distribution (main network) and a secondary function: c – penetration (secondary network). Hence, this segment should be classified as a D category segment, that is "*urbana di scorrimento*" (transit urban road). However, some discrepancies arise from the comparison with the Italian standards.

Tab. 14.7: Road categories (name and ID), context, posted speed limit, number of lanes, lower and upper design speeds. (D.M. 6792/2001[3]).

						Design speed range	
Road Type According to the Code	*Id*	*Context*		*Posted Speed*	*Number of Lanes per Direction of Travel*	*Lower Design Speed (km/h)*	*Upper Design Speed (km/h)*
1	2	3		4	5	6	7
Transit Urban	D	Urban	main road	70	2 or more	50	80
			possible service road	50	1 or more	25	60
Neighbourhood Urban	E	Urban		50	1 or more	40	60

First of all, a D road should have a speed limit of 70 Km/h but Viale Papa Giovanni XXIII has a speed limit of 50 Km/h.

Tab. 14.8: Road category (name and ID), context, lane width, minimum traffic island width and shoulder width. (D.M. 6792/2001[3]).

Road Type According to the Code	*Id*	*Context*		*Standard Lane Width (m)*	*Standard Traffic Island Width (m)*	*Standard Left Shoulder Width (m)*	*Standard Right Shoulder Width (m)*	*Standard Emergency Lane Width (m)*
1	2	3		8	9	10	11	12
Transit Urban	D	Urban	main road	3.25	1.8	0.50	1.00	-
			possible service road	2.75	-	0.50	0.50	-

Further comments can be made as based on the comparison of the road Viale Papa Giovanni XXIII with the standardized characteristics of a D road: the minimum requirements for the number of lanes (2 or more) and the traffic island width are met. However, in this case, both the requirements about lane and minimum shoulder widths (always smaller than 50 cm) are always not met. Moreover, the D.M. 6792/2001[3] defined the on-street parking, pedestrians and heavy vehicles interaction with the road.

Tab. 14.9: Road category (name and ID), context, requirements for on-street parking, public transport, pedestrian traffic, accesses (D.M. 6792/2001[3]).

Road Type According to the Code	*Id*	*Context*		*Parking Regulations*	*Public Transport*	*Pedestrian Traffic*	*Accesses*
1	2	3		18	19	20	21
Transit Urban	D	Urban	main road	Allowed in separated spaces with concentrated entries and exits	Reserved lane and/or special stop areas	On protected sidewalks	Not allowed
			possible service road	Allowed in ad-hoc spaces (parking lots)	Stop areas	On sidewalks	Allowed

The D.M. 2001[3] requirements for on-street parking, public transport lanes and pedestrian flows do not match the characteristics of Viale Papa Giovanni XXIII where: the on-street parking is allowed, the pedestrian flow on sidewalk is without protections and the public transport is mixed with the vehicular flow.

Hence, the function of the road should be considered regardless of its geometry, since urban roads have been mostly designed before the publication of current standards. They often do not follow requirements and they do not take into account all the possible city development scenarios. In fact, Viale Papa Giovanni XXIII is one of the most important roads in the city of Bari and the social, demographic and economic development of the city has influenced the road function. Nowadays, this road is a main arterial road for the city. Moreover, the presence of universities, offices, commercial activities and schools cause congestion on the road, mainly due to commuter traffic. At the end of these analysis, Viale Papa Giovanni XXIII was labelled as a E category road ("urbana di quartiere", neighbourhood urban road), in respect of all the standard requirements.

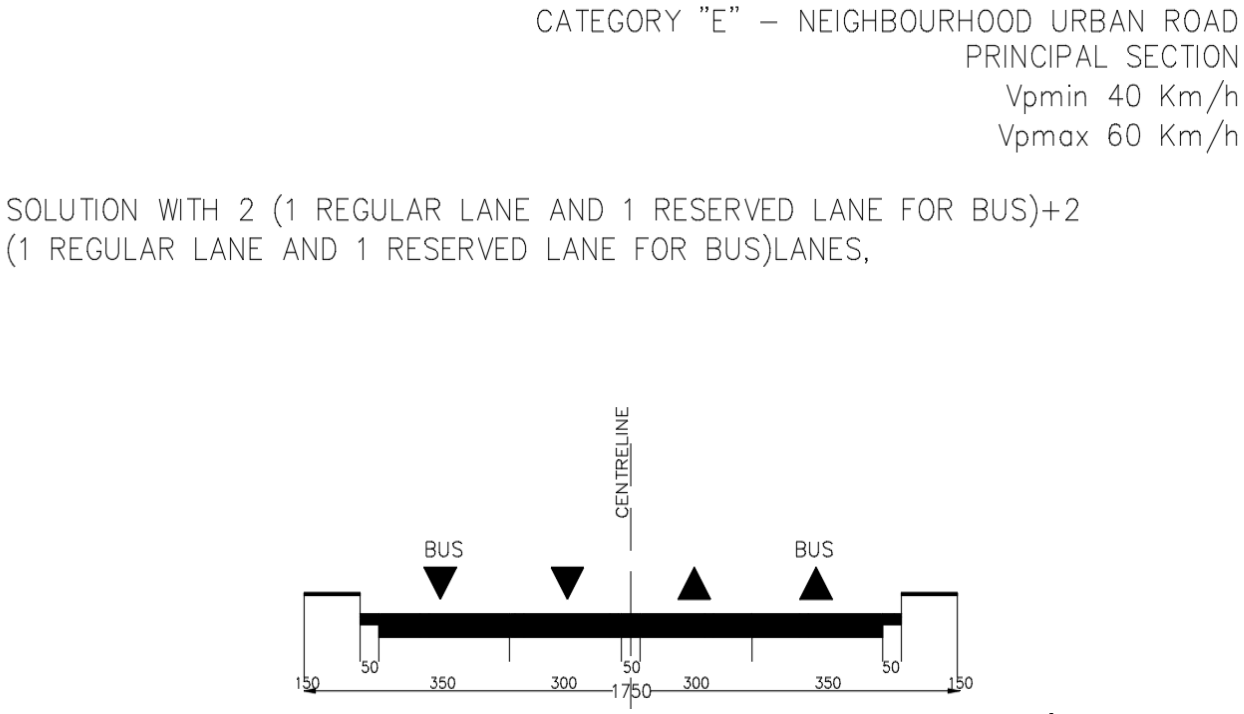

Fig. 14.22: Standard cross section of the "E" category road according to D.M. 6792/2001[3].

Tab. 14.10: Road category (name and ID), lane and minimum shoulder width (D.M. 6792/2001[3]).

Road Type According to the Code	*Id*	*Context*	*Standard Lane Width (m)*	*Standard Traffic Island Width (m)*	*Standard Left Shoulder Width (m)*	*Standard Right Shoulder Width (m)*	*Standard Emergency Lane Width (m)*
1	2	3	8	9	10	11	12
Neighbourhood Urban	E	Urban	3.00	-	-	0.50	-

Via Papa Giovanni XXIII is not provided with shoulders, because they are replaced by on-street parking spots.

Tab. 14.11: Road category (name and ID), context, requirements for: on-street parking, public transport, pedestrian traffic, accesses (D.M. 6792/2001[3]).

Road Type According to the Code	*Id*	*Context*	*Parking Regulations*	*Public Transport*	*Pedestrian Traffic*	*Accesses*
1	2	3	18	19	20	21
Neighbourhood Urban	E	Urban	Allowed in ad-hoc spaces (parking lots)	Stop areas or possible reserved lane	On sidewalks	Allowed

Also, the on-street parking spots on Via Papa Giovanni XXIII are allowed by the D.M. 6792/2001[3]. The same consideration is valid for the accesses. According to these standard requirements, Viale Orazio Flacco was categorized as an E category road as well, since Viale Orazio Flacco matches all requirements, except for shoulders which are 0.25 m wide (instead of 0.50 m, as shown in table 14.4). The cross sections of this street are shown below.

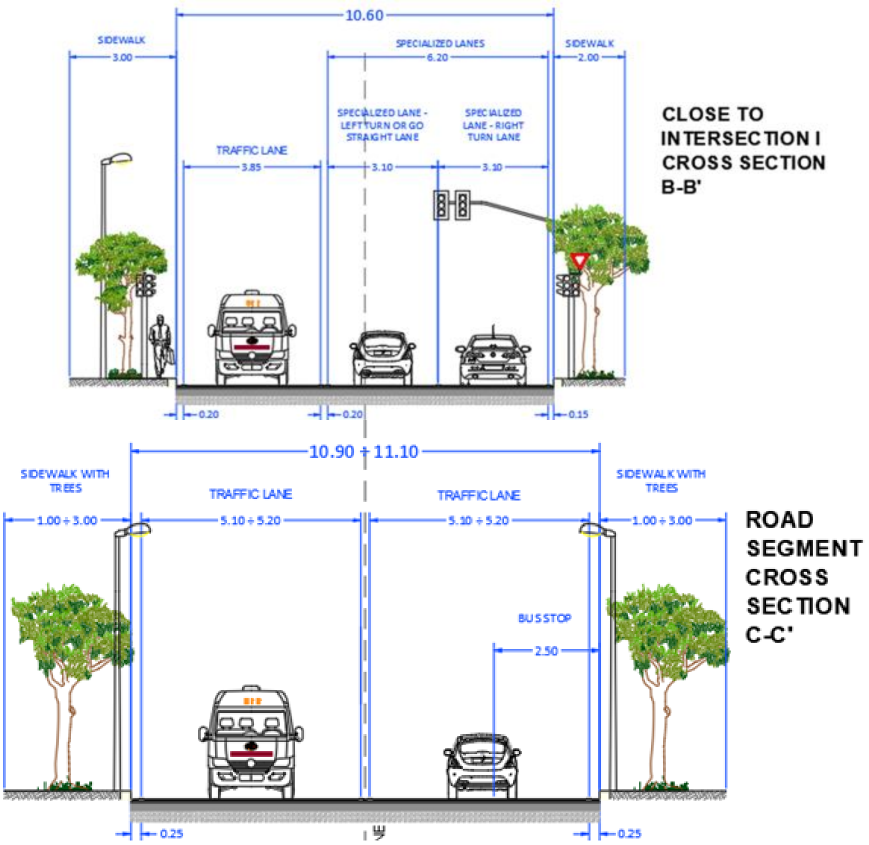

Fig. 14.23: Cross section of Viale Orazio Flacco.

It is impossible implementing a reserved bus lane due to the road width, which is constrained by some physical aspects, like buildings or ancient trees. Via Saverio Poli was categorized as a F category road; its cross sections are shown below.

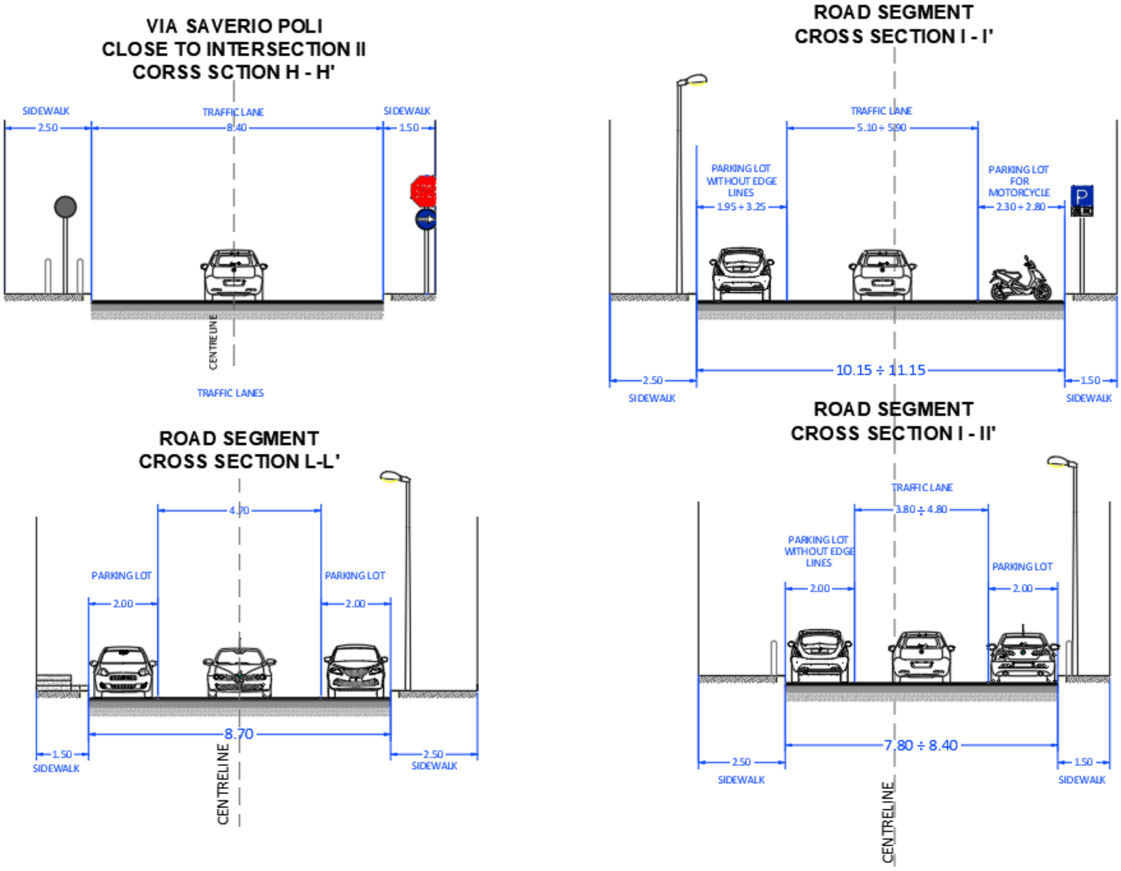

Fig. 14.24: Cross section of Via Saverio Poli.

All the characteristics of a F category road stated in the D.M. 6792/2001[3] are met.

Tab. 14.12: Road categories (name and ID), context, posted speed limit, number of lanes, lower and upper design speeds. (D.M. 6792/2001[3]).

						Design speed range	
Road Type according to the Code	*Id*	*Context*		*Posted Speed*	*Number of Lanes per Direction of Travel*	*Lower Design Speed (km/h)*	*Upper Design Speed (km/h)*
1	2	3		4	5	6	7
Local	F	Rural	F1	90	1	40	100
			F2	90	1	40	100
		Urban		50	1 or more	25	60

Tab. 14.13: Road category (name and ID), context, lane width (D.M. 6792/2001[3]).

Road Type according to the Code	*Id*	*Context*		*Standard Lane Width (m)*	*Standard Traffic Island Width (m)*	*Standard Left Shoulder Width (m)*	*Standard Right Shoulder Width (m)*	*Standard Emergency Lane Width (m)*
1	2	3		8	9	10	11	12
Local	F	Rural	F1	3.50	-	-	1.00	-
			F2	3.25	-	-	1.00	-
		Urban		2.75	-	-	0.50	-

Tab. 14.14: Road category (name and ID), context, requirements for: on-street parking, public transport, pedestrian traffic, accesses (D.M. 6792/2001[3]).

Road Type according to the Code	*Id*	*Context*		*Parking Regulations*	*Public Transport*	*Pedestrian Traffic*	*Accesses*
1	2	3		18	19	20	21
Local	F	Rural	F1 F2	Allowed in lay by	Stops arranged in special areas on the carriageway side	On shoulder	Allowed
		Urban		Allowed in ad-hoc spaces (parking lots)	Stop areas	On sidewalks	Allowed

The D.M. 6792/2001[3]. standard states "*in case of a road with one lane and so one direction, the sum of the width of lanes and shoulders must be 5.50 m, increasing the lane width up to 3.75 m and the available width difference should be implemented on the right shoulder*". This characteristic is matched as well, even if in the middle of the segment, the lane has a width of 3.80 m with on-street parking spots (marked with lines). The presence of on-street parking spot markings replaces the shoulders. The same category of road, F, was assigned to Via Niceforo, whose cross sections are represented below.

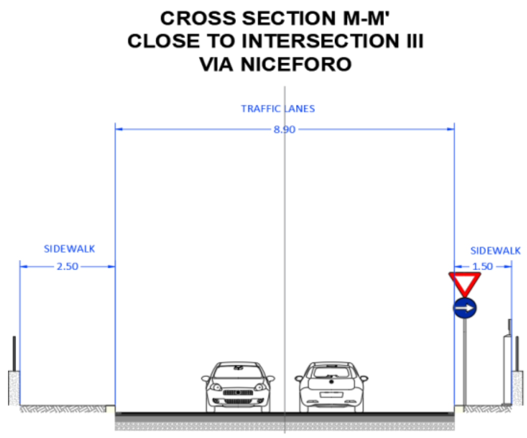

Fig. 14.25: Cross section, intersection II – Via Niceforo.

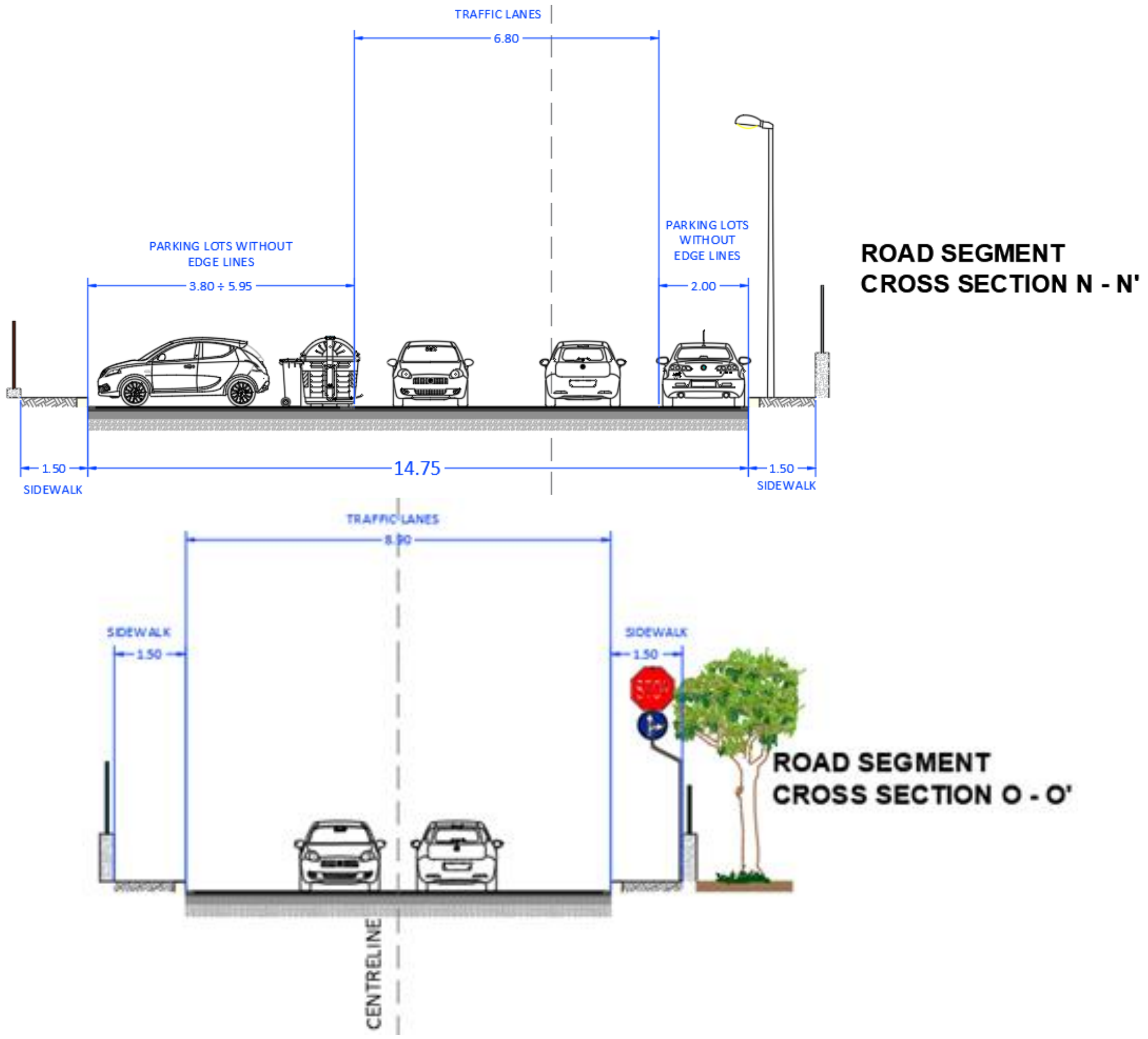

Fig. 14.26: Cross section, road segment sections – Via Niceforo.

Most of the requirements are also met for Via Niceforo, except for the on-street parking spots which are not drawn but the parking is allowed. Allowing parking reduces the lane width to a minimum of 2.65 m (considering the width of the parking area 1.80 m per each side, so 3.60 m in total), while the minimum suggested by the D.M. 6792/2001[3]) is 2.75 m. The shoulders are absent here too.

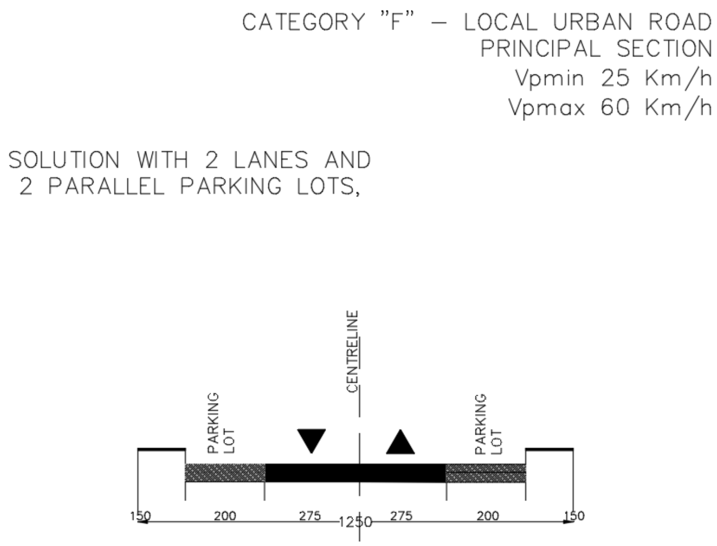

Fig. 14.27: Standard cross section of the F category road according to D.M. 6792/2001[3].

14.5 Diagnosis

14.5.1 Inspections and assessment of site conditions (D.Lgs. 35/2011[4])

Legislative Decree 35/2011 "*Guidelines for the Safety Management of Road Infrastructure*"[4] defines the inspection programme. The guidelines are the transposition of the "*European Directive on Safety Management*" into Italian law.

These inspections must be considered as a preventive intervention. They must assess segments where anomalous data in terms of both observed and expected crashes are detected. Therefore, the objective of inspections in an urban site is to analyse, characterise and record the "typical" experience of a user approaching the site, with or without a vehicle. Inspections, moreover, are carried out both in specific sites and along all the homogeneous segments to highlight the presence of anomalies, peculiarities and problems.

Within the decree, different types of specific inspection sheets are included in the Guidelines for different cases. However, not all the parameters concerning the urban environment (which are used for the design) are considered. Therefore, new inspection sheets have been developed on the basis of those provided by the Guidelines. They take into account new parameters that may affect safety in the urban area. Inspections on homogeneous segments have been carried out in both directions and both left and right sides have been analysed for each travel direction. In addition, road inspections were progressively carried out at every 50 or 100 m section (depending on the case) of the road segments, to ensure a better scanning of problems over the segments.

With regards to the intersections, each intersection was divided into quadrants: the centre and the different portions of segments which merge in the intersection itself (see figures below). The "centre" quadrant is the portion of the intersection which is delimited by the segment quadrant boundary lines; this boundary line is conventionally assumed as the horizontal stop sign or the farther point of the pedestrian crossing from the intersection centre. Where these elements are absent, an intersection quadrant is the area delimited by the boundary line of the intersection, which could be an artificial line placed 5 meters beyond the curb. The intersection division into quadrants is shown below.

Fig. 14.28: Intersection I quadrants (photo source Google Earth). The area is divided in quadrants: Sud = Southern quadrant; Est = Eastern quadrant; Nord = Northern quadrant; Ovest = Western quadrant; Centro = Central quadrant (photo source Google Earth).

[4] Ministerial Decree n. 137 of 2 May 2012, *Guidelines for the management of road infrastructure safety pursuant to art. 8 of Legislative Decree no. 35 of 15 March 2011 - Linee guida per la gestione della sicurezza delle infrastrutture stradali ai sensi dell'art. 8 del decreto legislativo 15 marzo 2011, n. 35*.

Fig. 14.29: Intersection II and Intersection III quadrants (photo source Google Earth). The area is divided in quadrants: Sud = Southern quadrant; Est = Eastern quadrant; Nord = Northern quadrant; Ovest = Western quadrant; Centro = Central quadrant (photo source Google Earth).

Fig. 14.30: Intersection IV quadrants (photo source Google Earth). The area is divided in quadrants: Sud = Southern quadrant; Est = Eastern quadrant; Nord = Northern quadrant; Ovest = Western quadrant; Centro = Central quadrant (photo source Google Earth).

The used inspection sheets rely on macro-items: road, signs, access, road pavement, lighting and other aspects. There are also sub-items: signs, markings, vertical signs and presence of traffic lights have been specified. For all inspection sheets, the presence or absence of the element was firstly indicated and then a judgement was provided as follows:

- if there were no problems, the line was left blank;
- if the detected problems are averagely severe, the box was highlighted in yellow;
- if the problems detected are very severe, the box was highlighted in red.

For a complete analysis of road network conditions, the inspection sheets have been completed by carrying out inspections both during daytime and night-time, indicating in each used sheet, the day and time when the individual inspection was carried out. The results of the inspections are presented below.

Tab. 14.15: Example of inspection sheet for Viale Papa Giovanni XXIII, from Intersection I to Intersection IV (heading to Intersection IV) – part 1.

Inspection modalities	*Daytime*	*Night-Time*	*Date*	*Starting Time*	*End Time*
1st inspection			21/05/2018	16:06	17:46
2nd inspection			11/06/2018	20:45	22:06

					Left Side				*Right Side*			
Macro-item	*Item*	*Parameter*	*Metric*	*J**	*0.10*	*0.20*	*0.30*	*0.40*	*0.10*	*0.20*	*0.30*	*0.40*
Road	*Roadway*	*Side Shoulder*	*absence or insufficient width*	M								
				S								
			narrowing close to attractive areas for pedestrians	M								
				S								
		Right Lane	*insufficient width*	M								
				S								
			inadequacy of coordination with other road users	M								
				S								
		Specialized Lane	*insufficient width*	M								
				S								
			inadequacy of coordination with other road users	M								
				S								
		Reserved Lane	*insufficient width*	M								
				S								
			inadequacy of coordination with other road users	M								
				S								
		Public Transport Stops	*inadequate dimensions*	M								
				S								
			location	M								
				S								
			unconformity with other pedestrian paths	M								
				S								
		Median	*absence*	M								
				S								
			negative effects on sight distance	M								
				S								
			inadequacy of the space layout	M								
				S								
		Lateral Parking Lots	*inadequacy of the space layout*	M								
				S								
			inadequacy of coordination with other road users	M								
				S								
		Cycle-Path	*section width*	M								
				S								
			pavement maintenance	M								
				S								
			presence of obstacles	M								
				S								
			flow separation	M								
				S								
	Road Edges	*Sidewalks*	*inadequate height*	M								
				S								
			presence of obstacles	M								
				S								
			inadequacy for people with disabilities	M					✓			
				S						✓		
			insufficient width	M								
				S								
			pavement conditions	M					✓	✓		
				S								
			pavement type	M	✓	✓						
				S								

*Judgment provided: M = medium danger according to the detected conditions; S = severe danger according to the detected conditions.

*Tab. 14.16: Example of inspection sheet for Viale Papa Giovanni XXIII, from Intersection I to Intersection IV – part 2**.*

					Left side				*Right side*			
Macro -item	*Item*	*Parameter*	*Metric*	*J**	*0.1*	*0.2*	*0.3*	*0.4*	*0.1*	*0.2*	*0.3*	*0.4*
Road Signs	*Horizontal Signs*	*Edge Line Visibility*	*insufficient retro-reflectivity*	M								
				S								
		Lane Lines Visibility	*insufficient retro-reflectivity*	M								
				S								
			Inadequacy for the steering manoeuvre	M								
				S								
		Specialized Lane Lines Visibility	*insufficient visibility of edge lane lines*	M								
				S								
			insufficient visibility of dir. arrows	M								
				S								
		Pedestrian Crosswalks	*insufficient visibility*	M								
				S								
			absence	M								
				S								
		Cycle Crosswalks	*insufficient visibility*	M								
				S								
			absence	M								
				S								
	Vertical Signs	*Danger/ prescription/ indication Signs*	*insufficient visibility*	M								
				S								
			not legible	M								
				S								
			not intelligible	M								
				S								
			interferences with obstacles	M								
				S								
	Light Signals	*Traffic Lamps*	*inadequacy of the phases*	M								
				S								
			inadequate location	M								
				S								
			absence	M								
				S								
			insufficient maintenance	M								
				S								
		Danger/ prescription Signs	*insufficient maintenance*	M								
				S								
		Variable Message Sign	*insufficient intelligibility*	M								
				S								
Accesses	*Private accesses*	*Location*	*inadequacy*	M								
				S								
		Sight distance	*inadequacy*	M					✓			
				S						✓		
Pavements	*Surface course*	*Deformations (potholes, cracks)*	*presence*	M								
				S								
		Drainage	*inefficient maintenance*	M								
				S								
		Friction	*inadequacy*	M								
				S								
	Curbs	*Discontinuity*	*inadequacy*	M								
				S								
	Tram railroad	*Discontinuity*	*inadequacy*	M								
				S								
	Manholes, covers, drains	*Vertical gradient*	*inadequacy*	M								
				S								
		Discontinuity (longitudinal/cross)	*inadequacy*	M								
				S								

*Judgment provided: M = medium danger according to the detected conditions; G = severe danger according to the detected conditions.
**Other macro-items not shown here are: lighting system (items: spread, considering absence or maintenance, or punctual at intersections or pedestrians/cyclists crosswalks, considering absence or maintenance) and other factors (items: public service such as parking lots, bus stops, crosswalks; land occupation; traffic calming, considering chicanes/lane narrowing, raised intersections, mini roundabouts; billboards both on segments and at intersections; other safety devices such as speed detectors, speed reducers or bumps; always considering their possible inadequacy).

14.5.1.1 Description of the detected problems

Viale Papa Giovanni XXIII shows pavement damages and sidewalk inefficiencies in respect to disabled people (e.g. absence of ramps). There are no horizontal pedestrian crossing signs on the secondary road at the intersection with Via Papa Bonifacio IX and there are no vertical pedestrian crossing signs to point out the crosswalk on Viale Papa Giovanni XXIII.

In proximity to the turning lane, close to the Intersection 1, it is possible to notice the presence of bad pavement conditions, which also imply a reduced visibility of markings (arrow signs) that indicate the turning lane manoeuvres.

Fig. 14.31: Bad pavement condition on the turning lane in Viale Papa Giovanni XXIII, before the intersection with Viale O. Flacco.

Further problems of this segment are related to the presence of accesses, which very often are not highly visible because of illegally parked vehicles which may obstruct the view. In addition, the median is characterised by the continuous presence of billboards (every 8.00 m) which could induce distracted driving. Finally, these billboards are characterised by a height not fully compatible with pedestrians, even though the median is not a pedestrian zone. As far as Viale Orazio Flacco is concerned, the presence of an insufficient shoulder (as previously described in the cross-section comparison with the standard cross sections) is evident. Moreover, the shoulder is often occupied by tree roots which can pose dangers for road safety.

Fig. 14.32: Tree roots presence in the shoulders of Viale Orazio Flacco (near the intersection with Via Storelli).

In addition, trees make difficult for people with disabilities to walk on the sidewalk. In some places, the sidewalk narrows, thus not allowing people with disabilities to easily pass by. The pavement is in poor conditions, which could cause problems for pedestrian mobility.

Fig. 14.33: Manhole cover at the bus stop.

In correspondence of the turning area near the intersection with Viale Papa Giovanni XXIII, there are some potholes, ruts and cracks. On this segment, the bus stop area does not have any reserved space. In fact, the bus stops on the driving lane.

There is no pedestrian crossing at this bus stop and this absence could lead to dangerous situations for pedestrians who need to cross the street after getting off the bus. In addition, there is a raised manhole cover (Figure 14.33) near the bus stop which could cause problems.

Via Niceforo, as well as other segments of the road network, does not have shoulders, because on-street parking is permitted on the roadside, but it is not indicated by ad-hoc markings.

Furthermore, it should be noted that, as previously stated, Via Niceforo has a road width which is slightly lower than the minimum required for F category roads (2.75 meters).

Even in this case, the sidewalks are not suitable for people with disabilities because there are no ramps, and the sidewalk is characterized by a marked irregularity due to the presence of accesses along the entire segment. In addition, the pavement of sidewalks is characterized by disruptions due to brushwood.

As far as the road pavement is concerned, there are slight cracks in the area near the intersection III, while the rest of the segment shows bad pavement conditions due to the presence of tree roots adjacent to the road.

Finally, there is the need to delimit the area used for urban waste bins because they may be moved in order to carve out new parking spots. In this way, the bins occupy a portion of the lanes.

As far as Via Poli is concerned, it is possible to highlight the inadequacy of the sidewalk for people with disabilities because, on the left side, the sidewalk is very high. The whole street is not provided with ramps and the pavement is characterized by disruptions.

Fig. 14.34: Municipal waste bins not placed in their stalls

Fig. 14.35: Sidewalk elevation of Via Poli

Finally, along the segment there are slight pavement deformations. The first intersection described is the intersection between Viale Papa Giovanni XXIII, Viale Orazio Flacco, Viale Cotugno and Viale Pio XII which is called Intersection I for the sake of simplicity.

The intersection I, is characterized by a lack of shoulders, a problem probably linked to the recent renovation of the wearing course, for which the final judgement is postponed to a subsequent inspection.

As far as the pavements are concerned, firstly, there are obstacles such as the municipal police booth near the South quadrant and a pole on the traffic island (West quadrant).

Again, the sidewalks are inadequate for people with disabilities due to the widespread absence of ramps. The pavement of the sidewalks is not optimal, because some deformations and potholes are present.

The North quadrant area is completely devoid of transverse give-way signs. Moreover, in the same point, there is lack of visibility of the vertical give-way sign, because it is covered by some tree branches.

With regard to the state of the pavement of the roadway, it can be noted that the entire intersection has recently been affected by a recent renovation of the wearing course, except in the area of the North and South quadrants. In the latter quadrants, the presence of deformations such as cracks and potholes are evident, Figure 14.36.

Fig. 14.36: Pavement deformations at the North quadrant.

There is a difference in the elevation of joints and manholes with respect to the road surface especially in the Eastern, Southern and Central quadrants.

As far as the intersection between Viale Papa Giovanni XXIII and Via Poli (called Intersection II for the sake of simplicity) is concerned, in the North quadrant (Via Poli) there are no ramps for disabled people. In addition, the North quadrant sidewalks are characterized by a width of 1.5 m, which is not sufficient for the presence of curved poles and commercial activities with shop windows where pedestrians could stop. The pavement, made of tiles, has some areas with disruption; in addition, the median is made of asphalt mixture.

The horizontal stop sign, in the North quadrant, is not very visible due to the presence of bad pavement conditions. The North quadrant is also characterized by the presence of faded pedestrian crossings and absence of vertical signs for the pedestrian crossing. In both the East and West quadrants vertical signs for the pedestrian crossing are not present.

In the North quadrant, bad road conditions and temporary road works reduce the visibility of markings. The remaining part of the intersection is in good conditions: asphalt pavement was recently renovated along Viale Papa Giovanni XXIII.

Finally, with regard to joints and manholes a slight difference in elevation in the North and Centre quadrants is present.

In the South quadrant of the intersection between Viale Papa Giovanni XXIII and Via Niceforo (defined as Intersection III), ramps are completely absent. In the East and West quadrants of the same intersection, the sidewalk (even having an adequate width), has some instable tiles. In the South quadrant, there is the presence of both instability and potholes due to the removal of some tiles. The median is made of asphalt mixture.

Regarding pedestrian crossings, in the East and West quadrants, there are no appropriate vertical signs for pedestrian crossings; while in the South quadrant both vertical and horizontal signs (pedestrian crossings) are completely absent.

The pavement of the roadway is characterized by slight cracks and potholes in the South quadrant. In the remaining quadrants, the road pavement is in good conditions because recently renovated.

Regarding the intersection between Viale Papa Giovanni XXIII, Via Lioce and Via Giovene (called

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Intersection IV), shoulders are partially absent in the South quadrant. The ground-level traffic islands in the Centre quadrant painted with markings may lead to situations in which drivers do not respect the organization of lanes, turning in a chaotic way in any traffic condition at the intersection.

There is an inadequate elevation of sidewalks at the North and East quadrants. In the portion of the sidewalk that falls in the Centre quadrant there is an obstacle, that is a pole, perhaps previously part of advertising signs, from which the advertising panel has been removed.

For the whole intersection, except for the West quadrant, there is a serious inadequacy for disabled people due to the widespread lack of ramps (North and South) and pavement damages.

The sidewalk width near to the booth is inadequate because the distance between it and the edge of the sidewalk is less than 1.50 m that is the minimum value accepted by the D.M. 6792/2001[3].

The South quadrant sidewalk width is 1.5 m wide, but it is partly occupied by poles and telephone and electrical service stations.

The sidewalk surface close to the North, South and West quadrants is characterized by some issues. In the portion of sidewalk that falls in the East and Centre quadrants there are irregularities, potholes and portions of steel poles protruding out of the surface, becoming dangerous. In addition, the sidewalk is made of asphalt mixture in the South and Centre quadrants, while there are some concrete portions in the West quadrant.

Pedestrian crossings are not signalized in the North and South quadrants, thus being scarcely visible.

With regard to the road pavement, there is an optimal condition for most of the intersection recently renovated (which involved Viale Papa Giovanni XXIII too) with the exclusive presence of slight cracks and potholes concentrated in the North quadrant area (not influenced by the renovation).

Joints and manholes are slightly different in elevation than the ground level, but this happens only in the East quadrant.

The presence of road markings for parking was also evaluated because in the East quadrant there are parked vehicles but no marked parking lots as well as in the West quadrant (Via Lioce side), leading to illegal parking.

Finally, there is an area delimited through horizontal signs for positioning waste bins, but they are often out of position because drivers often stop their vehicles in this area.

14.5.1.2 Road network lighting system

The inspections carried out during night (11/06/2018 at about 10 p.m.) show that the entire road network analysed is correctly illuminated with both diffused lighting and spotlights at intersections.

However, there are maintenance problems related to some malfunctioning lamps in Viale Papa Giovanni XXIII as well as some other lamps in Viale Orazio Flacco (near the bus stop).

Fig. 14.37: Lighting conditions, Viale Papa Giovanni XXIII.

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Fig. 14.38: Lighting conditions, Viale Orazio Flacco.

14.5.2 Visibility checks

Visibility at an intersection is the fundamental requirement for any vehicle user, to be able to safely travel. In order to guarantee the regular functioning of at-grade intersections, it is necessary to define the manoeuvres of the vehicular flows in the node and to divide them into main and secondary flows. The regulations prescribe that traffic lights, give-way or stop signs should be implemented in order to avoid intersections without signal regulation and therefore, simple give-way to the right.

It is necessary that intersections are organised in such a way that visibility is always ensured so that drivers can see the presence of any obstacles or other vehicles in their trajectory and can be able to react immediately, by selecting the manoeuvre type to be carried out.

In order to achieve this aim, an obstacle-free area, the triangle of visibility (Figure 14.39), is defined so that two vehicle drivers travelling on two different directions can see each other. The visibility triangle dimensions are set as a function of the travelling speed on the primary and secondary segments. Once the potential point of collision has been defined as the intersection of the trajectories travelled by two vehicles, in order to be able to see each other, they must be at an appropriate distance, as indicated in Figure 14.39. D_{f1} and D_{f2} are respectively the distances from the potential point of collision for the vehicle travelling in the main flow and the driver in the secondary flow.

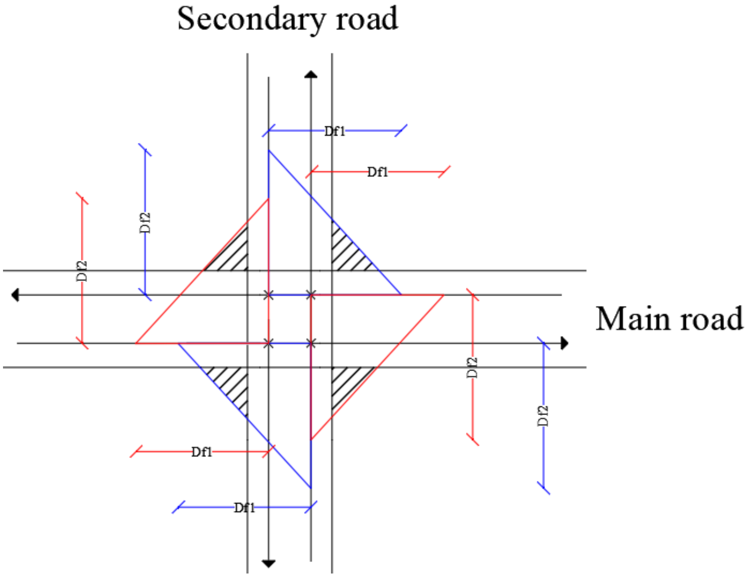

Fig. 14.39: Visibility triangles.

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The length of the longest side of each triangle is called the main sight distance, indicated by D and it is computed as the speed v, multiplied by time t, where:

- v is the reference speed, expressed in [m/s], equal to the value of the characteristic design speed of the section or the posted speed limit (if present);
- t is the manoeuvring time, expressed in [s], between the approach of the vehicle in the area of visibility of the intersection and the end of the manoeuvre in the point of collision. It takes the value of 12 s for manoeuvres governed by the give-way sign, and 6 s for those governed by the stop sign.

The length of the short side depends on the intersection sign setting:

- in the case of a give-way sign: the short side of the visibility triangle is at a distance of 20.00 m from the main road;
- in case of presence of the stop sign, the short side will be 3.00 m from the stop line.

In the case of the investigated road network, the only intersection regulated by traffic lights is the intersection I (i.e., the intersection between Viale Papa Giovanni XXIII, Viale Orazio Flacco, Viale Cotugno and Viale Pio XII); while the intersection II (between Viale Papa Giovanni XXIII and Via Poli) is regulated by the stop sign; the intersections III (Viale Papa Giovanni XXIII and Via Niceforo) and IV (Viale Papa Giovanni XXIII int. Via Lioce) are regulated by the give-way sign.

A careful analysis of crash data suggests the need to define the visibility triangles also for intersections regulated by traffic lights, because crashes have occurred also during the hours when the traffic lights are flashing or off, i.e. between 11:00 p.m. and 7:00 a.m.

The sign that regulates intersection I when the traffic lights are off is the give-way. The following values have been considered:

- design speed equal to 50 Km/h (i.e., 13.89 m/s);
- at the give-way sign, the time (t) is set equal to 12 s, with the short side of visibility triangle equal to 20.00 m from the main road;
- at the stop sign, the time (t) is equal to 6 s, with the short side of visibility triangle equal to 3 m from the stop line;
- the greater side of visibility triangle is given by the following equations;

$D= v\times t$= 13.89 m/s × 12 s= 166.67 m in case of give-way (Eq. 14-1)

$D= v\times t$= 13.89 m/s × 6 s = 83.34 m in case of stop (Eq. 14-2)

- Viale Papa Giovanni XXIII plays the role of main street for both its functional and geometric characteristics, compared to the other roads composing the network under investigation.

The following figures show the visibility triangles drawn for the intersections under investigation.

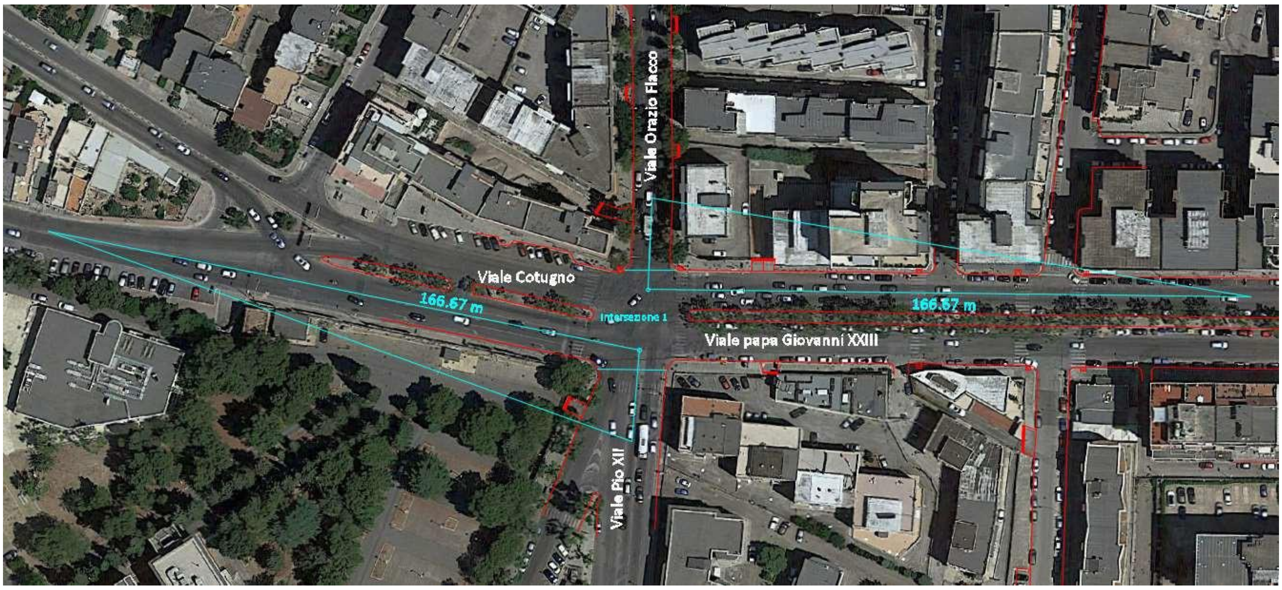

Fig. 14.40: Intersection I Visibility triangles (photo source Google Earth).

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Fig. 14.41: Intersection II Visibility triangles (photo source Google Earth).

Fig. 14.42: Intersection III Visibility triangles (photo source Google Earth).

Fig. 14.43: Intersection IV Visibility triangles (photo source Google Earth).

In particular, the Intersection IV sight distance, in correspondence of the traffic island, has also been determined for vehicles coming from Via Lioce that cross the first conflict zone with Viale Papa Giovanni XXIII, then occupying the intersection. Even if in this case there is a give-way sign, the vehicle is assumed to arrive in the proximity of the intersection with a lower speed than at the end of Via Lioce. Therefore, the short side of the visibility triangle will be equal to 3.00 m from the stop line (differently than what normally happens for the intersections regulated by give-way signs).

14.5.3 Analysis of crash data and crash location

The fundamental part of the diagnosis is certainly the study of data related to crashes that occurred in the past, which are divided according to different types, severity, environmental conditions and location. The crash data that are used in this project are from the ASSET-ISTAT database which are available thanks to the Municipality of Bari help. According to this, it was possible to find information on crashes occurred into the area covered by this study, from 2012 to 2016. A careful analysis of the data may allow to identify possible causes related to the period of the day, weather conditions or driver behaviour. In order to study crashes occurred on the different segments under study, the database provided by the Municipality of Bari, which contained all the necessary information, was examined. In order to read correctly the same database, the legend for the crash reports, by ASSET-ISTAT, was necessary. It helps to understand the information. The first data are about the location of the crash, obtained from:

- latitude (Y) and longitude (X) coordinates.
- information on the location of the crash specifically, dividing whether the crash occurred,
  - at an intersection: intersection, roundabout, signalized intersection, intersection with traffic lights unsignalized intersection and intersection type;
  - not at intersection: tangent, curve, another specific road element, evident slope, tunnel.
- indication of the name of the intersecting streets, and in some cases also of the house numbers.

After having localized all the crashes, the ones not falling in the area of study were not considered. From this procedure, a total number of 37 crashes occurred in the area of study. Although, there were particular cases in which a more careful analysis of the crash dynamics was necessary in order to univocally define the crash location, once the accidents had been located, a summary table was then filled, including other information contained in the accident database provided by ASSET-ISTAT, such as:

- localization, specifying both the “macro” and “specific” localizations.
  In detail, the “macro” location has been divided into:
  - segment;
  - 4-legged signalized intersection;
  - 3-legged signalized intersection;
  - 3-legged unsignalized intersection;

  The “specific” localization identifies:
  - the intersections affected by crashes;
  - the addresses for crashes occurred on segments;
- time information, the season;
  - the day of the week, indicating if it is a weekday or holiday/weekend. It should also be noticed that the indication of the weekend was used for those accidents occurred between 11.00 p.m. on Friday until 6.00 a.m. on Sunday;
  - the period of the day, identifying four-time windows: morning (6.00 a.m. -12.00 p.m.), afternoon (12.00 p.m.-6.00 p.m.), evening (6.00 p.m.-12.00 a.m.) and finally night (0.00 a.m. -6.00 a.m.)
- typical traffic conditions, whose information was obtained using the typical traffic function in Google Maps®. This function, depending on the day of the week and the time of day, returns the possible typical traffic condition when the crash occurred. Based on different colours of the function described above, four traffic categories were defined: smooth, medium, slow and congested traffic flow. The typical traffic values are detected within the 6:00 a.m. and 10:00 p.m. When the traffic data was not available because outside that time window, the ID “Data not provided” has been used. In addition, there have been cases where the typical traffic was not available for that particular road, so the designation “Data not provided” was used;
- the type of crash, which was indicated in a different colour in order to intuitively distinguish different types:
  - pedestrian collision (PC);
  - angle collision (AC);
  - sideswipe collision (SC);
  - rear-end collision (RC);
  - collision with fixed objects/animals (FC);
- vehicles involved, indicating the number of vehicles involved in each crash and the type of vehicles involved and, specifically for the examined crashes: car, moped, motorcycle, bicycle, urban/rural bus and emergency or police vehicle;

- presumed circumstances of the crash, indicating the sequence of events, liability of drivers and possible causes of the crash;
- weather conditions, specifically indicating in the examined crashes: clear sky, rain, fog or other conditions;
- pavement conditions, specifically indicating in the examined crashes: dry or wet;
- number of injuries and deaths.

An example of those information has been reported in Table 14.17. Crash data have been properly analysed, determining statistics useful to understand the recurrent problems of the investigated area. The data have been reported in Table 14.18.

Tab. 14.17: Examples of Information retrieved from crash data.

ID	*Macro-Localization*	*Season*	*Day*	*Moment of Day*	*Crash Type*	*N. of Involved Vehicles*	*Involved Vehicles*	*Crash Circumstances (Assumed based on the dataset)*	*Weather Conditions*	*Road Surface Conditions*	*Injuries*	*Deaths*
1	Rs	W	WD	M	PC	1	Sm	Irregular crossing manoeuvre by pedestrian	C	D	1	0
2	Rs	S	WD	M	RC	2	Q/Pv	Vehicle B travelled distracted or with unsafe behaviour	C	D	1	0
3	Rs	S	WD	A	RC	3	Pv/PvPv	Reduced headway by the Vehicle A	C	D	1	0
4	Rs	F	WD	E	PC	1	Sm	Irregular crossing manoeuvre by pedestrian	O	W	1	0
5	Rs	Sm	WE	A	FC	1	Pv	Vehicle A travelled distracted/with unsafe behaviour	C	D	1	0
6	Rs	F	WD	M	SC	3	Pv/Sm/Pv	High speed by Vehicle B	O	D	1	0
7	Rs	F	WD	A	AC	2	Pv/Sm	Vehicle B travelled distracted or with unsafe behaviour	C	D	1	0
8	Rs	W	WD	M	AC	2	Sm/Pv	Vehicle B did not respect the Stop sign	R	W	1	0
9	Rs	S	WD	A	RC	2	Pv/Sm	Vehicle B did not respect the Stop sign	C	D	1	0
10	Rs	F	WD	M	AC	2	Sm/Pv	High speed by Vehicle B	C	D	1	0
11	Rs	F	WD	E	AC	2	Mp/Mp	Vehicle B travelled distracted or with unsafe behaviour	C	D	1	0
12	4IL	W	WD	E	AC	2	Pv/Pv	Irregular steering manoeuvre by Vehicle B	C	D	1	0
13	4IL	W	WE	M	AC	2	Pv/Pv	Vehicle B did not respect the priority sign	C	D	2	0
14	4IL	W	WD	A	SC	2	Pv/Pv	Irregular left steering manoeuvres by Vehicle B	O	D	1	0

Macro-Localization: Rs: Road Segment; 4IL: Four Legs Intersection with Traffic Lights (specific localization was also available)
Season = F: Fall; W: Winter; S: Spring; Sm: Summer.
Moment of Day = A: Afternoon; E: Evening; M: Morning; N: Night.
Day = WD: Workday; WE: Weekend; FD: Feast Day.
Weather Conditions = C: Clear; O: Other; R: Rain
*Road Surface Condition*s = D: Dry, W: Wet.
Involved Vehicles: Pv: Private Vehicle; Sm: Solo Motorcycle; Mp: Motorcycle with Passenger; M: Moped; B: Bicycle; Ev: Emergency or Police Vehicle; Q: Quadricycle; Bf: Bus or “Filobus”.

Tab. 14.18: Summary of the detailed analysis of crash data part 1.

Variable	*N. of crashes*	*%*
Macro-localization		
3-legged intersection	1	2.7
4-legged intersection with traffic lights	18	48.7
4-legged intersection	7	18.9
Road segment	11	29.7
Specific localization		
Viale Papa Giovanni XXIII	7	18.9
Viale Orazio Flacco	2	5.4
Via Niceforo	2	5.4
Int. Flacco/GV XXIII/Pio XII	18	48.6
Int. GV XXIII/Poli	1	2.7
Int. GV XXIII/Lioce/Giovene	7	18.9
Season		
Winter	13	35.1
Spring	5	13.5
Summer	9	24.3
Fall	10	27.0
Type of day		
Working day	31	83.8
Weekend	4	10.8
Feast day	2	5.4
Crash type		
Pedestrian collision	6	16.2
Rear-end collision	7	18.9
Sideswipe collision	8	21.6
Angle collision	15	40.5
Collision with a fixed object	1	2.7
Main Circumstances		
Irregular crossing by pedestrian	2	5.4
Disrespect of the priority sign	1-2	4.0
Pedestrian impacted by a vehicle	2	5.4
Maneuvering vehicle	2	5.4
Disrespect of the headway	4	10.8
Distracted driving or unsafe driving	6	16.2
Not precise circumstance	1	2.7
Disrespect of the stop sign	6	16.2
High speed	4-5	12.2
Disrespect of the traffic light or of the policemen	3	8.1
Irregular steering	3	8.1
Overtaking at the intersection	1	2.7
Irregular overtaking maneuver by a pedestrian	1	2.7
Road surface conditions		
Dry	33	89.2
Wet	4	10.8
Weather conditions		
Clear	30	81.1
Rain	2	5.4
Snow	0	0.0
Other	5	13.5
Period of the day		
Morning (06:00 am - 12:00 pm)	14	37.8
Afternoon (12:00 pm - 06:00 pm)	12	32.4
Evening (06:00 pm - 12:00 am)	10	27.0
Night (12:00 am - 06:00 am)	1	2.7
Consequences	Number	Number per crash
Fatalities	0	0.0
Injuries	51	1.4

Tab. 14.19: Summary of the detailed analysis of crash data part 2.

Variable	*N. of crashes*	*%*
Vehicle in single-vehicle crashes		
Private vehicle	4	57.14
Motorcycle	2	28.57
Moped	1	14.29
Vehicle in two-vehicles crashes		
Private vehicle	1	3.85
Private vehicle/motorcycle	9	34.62
2 motorcycles with passenger	1	3.85
2 private vehicles	8	30.77
Private vehicle/bicycle	2	7.69
Bus or filobus/private vehicle	1	3.85
Private vehicle/emergency vehicle	1	3.85
Private vehicle/Moped	3	11.54
Vehicle in three-vehicles crashes		
3 Private vehicles	2	50.00
2 Private vehicles and 1 motorcycle	2	50.00

According to the crash analysis, that 49% of crashes are located at the intersection I (i.e., the intersection between Viale Papa Giovanni XXIII, Viale Orazio Flacco, Viale Cotugno and Viale Pio XII), the 30% of crashes is located on segments, while 19% of crashes is located at the intersection IV (i.e., the intersection between Viale Papa John XXIII, Via Lioce and Via Giovene).

Based on these data, it is possible to state that the majority of crashes actually occurred in the intersection I, which is essentially a 4-legged intersection with traffic lights. Only 2 out of 18 crashes have occurred when the traffic lights are off or flashing.

The winter crashes (35 %) prevail, while summer and autumn crashes are both around 25%. In addition, with regard to the type of day it is possible to assess that the majority of crashes (84%) occurred on weekdays. For this reason, it could be assumed that users involved in the crashes could be regular users of the analysed roads.

Regarding the period of the day: 38% of crashes occurred in the morning, 32% of them in the afternoon, 27% of crashes in the evening and only 3% of them at night. It is therefore not possible to define a time of day where crashes are mostly concentrated.

However, the majority of crashes have occurred in typical “medium” traffic conditions assumed by looking at the considered online (Google Maps®) source (57%), while only 27% of them occurred when traffic is slow or congested (as assumed by looking at the same source).

With regard to weather conditions, 81% of crashes occurred under clear sky conditions and only 5% under rainy conditions. This shows that the majority of crashes happened under optimum weather conditions, and only a small part of them under unfavourable conditions.

The pavement, in fact, was dry in the 89% of cases, while it was wet in the remaining 11%.

As far as the type of crash is concerned, 41% of the crashes occurred are angle collisions, 22% are sideswipe collisions, 19% are rear-end collisions while 16 % are pedestrian collisions. Finally, only one crash occurred with a parked vehicle.

In addition, in 16% of crashes, the causes are distracted or undecided driving behaviours and failure to comply with the stop sign. Moreover, in around 12% of cases, crashes were caused by high speeds, while around 11% of crashes occurred for failure to keep a safe distance.

From the analysis of these data related to the type and circumstances of crashes, it can be seen that crashes are mostly characterized by angle collisions that occurred at the intersections in the analysed area. These collisions could be very often correlated to a missed compliance with the vertical sign, as well as to distracted drivers. In this case, the use of smartphones and in-vehicle technologies while driving may lead to distracted driving and human errors. Thus, for instance, preventing driving distraction due to the use of mobile phones can be important for reducing distraction-related crashes.

Also, the rear-end collisions can be generally caused by high speeds and/or inappropriate headways. Pedestrian collisions are also important. They are located both at intersections (mostly in Intersection I) and on Viale Papa Giovanni XXIII.

Finally, as shown in table 14.19, all crashes that occurred in the area of study only caused injuries with different severities.

14.5.4 Crash frequency and rate

With the crash data provided and thus the 37 crashes, two fundamental measures can be defined with regard to road safety: the crash frequency and the crash rate.

These measures are referred to intersections but also to the individual segments which enter into the intersection itself. The crashes occurred in the intersection were counted solely as crashes at the intersections and not related to the segments entering into the intersections. Moreover, crash data belonging to the portion of the segments that have been previously included in the intersection area inspected (i.e., the previously defined quadrants on the segment) were considered for safety performance measures of intersections.

The frequency is defined as the number of crashes per unit of time that occur at a given site, while the crash rate can be assumed as the probability that, ceteris paribus, a vehicle may be involved in a crash while driving.

This probability is given by the expected crash frequency on a road section of a given length (or an intersection) divided by all the vehicles that may be involved (the exposure to crash). It follows that the crash rate represents the number of crashes occurring in a given period of time (years of observation) in relation to a particular exposure measure (crashes per million of vehicles on the carriageway). The crash rate is so given by the following formula (where the length is omitted for intersections):

$$Crash\ rate = \frac{N_{crashes} \times 10^6}{Years_{observation} \times AADT \times 365\ x\ Length} = \left[\frac{Crashes}{mlnvehicles\ x\ km}\right] \qquad \text{(Eq. 14–3)}$$

While the frequency represents a direct metric, the crash rate must be referred to the annual average daily traffic (AADT) for the whole year. At the intersections, the AADT is the result of the sum of flows entering the node, while in the generic segment the AADT is given by the sum of the flows for each travel direction. In case of missing traffic volumes, the AADT can be estimated as based on peak hour traffic surveys, thus converting peak volumes into AADT by means of empirical relationships. The available starting data did not provide the average daily traffic but the traffic for hourly peak flows. Starting from these data and then using the empirical correlation contained in Wolshon and Pande (2016)[5] that links the AADT to the volume of vehicles at rush hours, the AADT was calculated. Based on these empirical correlations, the peak hour traffic is generally between 7 and 12% of the AADT, so to get the AADT the peak hourly traffic was multiplied by 10.

For each element of the network, the following equations have been defined,

- Crash rate for the intersections (Rint):

$$Rint = \frac{N_{Crashes} \times 10^6}{Years_{observation} \times (peak\ flow \times 10) \times 365} \left[\frac{crashes}{mlnvehicles}\right] \qquad \text{(Eq. 14–4)}$$

- Crash rate for the segments (Rseg):

$$Rseg = \frac{N_{Crashes} \times 10^6}{Years_{observation} \times (peak\ flow \times 10) \times 365 \times km} \left[\frac{crashes}{mlnvehicles}\right] \qquad \text{(Eq. 14–5)}$$

- Crash frequency for both intersections and segments (Fint, Fseg):

$$Fint = Fseg = \frac{N_{Crashes}}{Years_{observation}} \left[\frac{crashes}{year}\right] \qquad \text{(Eq. 14–6)}$$

A crash outcome can have different degrees of severity.

The crash severity can be defined according to the KABCO scale, which defines five levels of severity, one for each letter of the acronym (K fatal injury, A incapacitating injury, B not incapacitating injury, C possible injury and O property damage only). Only personal injury crashes are reported in the ISTAT database while property damage only crashes are excluded, resulting in a KABC severity level on a KABCO scale.

In the following tables, it is possible to identify the partition of crashes between the relevant network elements (the crashes occurring at segments or intersections) and both the computed metrics *crash rate* and *crash frequency*.

[5] Wolshon B., Pande A. (2016), *Traffic engineering handbook*, John Wiley & Sons, Hoboken, New Jersey, USA.

Tab. 14.20: Computed crash rates and frequencies for the elements in the studied road network.

Network element	*Segment L [km]*	*Peak flow*	*N. of crashes*	*R [crashes/ (mln veh x km)]*	*R1 [crashes/ (mln veh)]*	*F [crashes/ (year x km)]*	*F1 [crashes/ year]*
Intersection 1		3556	17		0.262		3.4
Viale Orazio Flacco	Whole 0.138	1535	2	0.519	0.071	2.909	0.4
Viale Papa Giovanni XXIII	T1 0.090	1492	1	0.410	0.037	2.235	0.2
Intersection 2		842	1		0.065		0.2
Viale Papa Giovanni XXIII	T1 North 0.090	842	0	0.000	0.000	0.000	0.0
Viale Papa Giovanni XXIII	T2 North 0.113	750	3	1.947	0.219	5.331	0.6
Via Poli	Whole 0.086	92	0	0.000	0.000	0.000	0.0
Intersection 3		758	0		0.000		0.0
Viale Papa Giovanni XXIII	T1 South 0.090	650	1	0.942	0.084	2.235	0.2
Viale Papa Giovanni XXIII	T2 South 0.113	740	3	1.974	0.222	5.331	0.6
Via Niceforo	Whole 0.172	126	2	5.044	0.870	2.320	0.4
Intersection 4		2031	7		0.189		1.4
Viale Papa Giovanni XXIII	T2 0.113	1490	6	1.960	0.221	10.662	1.2

The survey time was decided on the basis of the indications obtained from the typical traffic detected from Google Maps®, in order to count vehicles during the peak hour.

In order to refer to counts of homogeneous vehicles, coefficients were applied (derived from ANAS) to derive equivalent passenger cars (for motorcycles, buses, trucks and vans).

The number of bicycles running through the network has been measured, but it has not been converted into equivalent vehicles, because they are not motor vehicles. They were counted anyway, because the cycling flow can indicate the need for introducing a cycle path.

In this way, the equivalent traffic was defined with reference to the half hour of detection. Then, this value was multiplied by 2 in order to consider the hourly traffic (peak flow). The AADT was then obtained by multiplying the peak flow by 10. Once the peak flows for the segments had been determined, the peak flow value at intersections was computed as the sum of the peak flows entering into the intersections from the adjacent segments.

The availability of traffic data allows computing the Origin - Destination Matrices for the different intersections.

All traffic volume data collected for segments and intersections are reported below.

Tab. 14.21: Example of traffic data survey and O-D Matrix - Intersection I.

		Destination			
		A	B	C	D
Origin	A		571	495	389
	B	293		51	200
	C	554	57	30 (cars making inversions)	101
	D	379	363	73	

From C to A	*N. of vehicles*	*N. of equivalent vehicles*
Car	515	515
Motorcycle	64	21.3
Bus	0	0
Truck	0	0
Lorry	0	0
Van	12	18
Bicycle	4	
		554.3 → 554

From C to B	*N. of vehicles*	*N. of equivalent vehicles*
Car	51	51
Motorcycle	0	0
Bus	0	0
Truck	0	0
Lorry	0	0
Van	4	6
Bicycle	4	
		57

From C to D	*N. of vehicles*	*N. of equivalent vehicles*
Car	85	85
Motorcycle	12	4
Bus	4	12
Truck	0	0
Lorry	0	0
Van	0	0
Bicycle	0	
		101

From D to B	*N. of vehicles*	*N. of equivalent vehicles*
Car	281	281
Motorcycle	60	20
Bus	12	36
Truck	8	20
Lorry	0	0
Van	4	6
Bicycle	4	
		363

From D to C	*N. of vehicles*	*N. of equivalent vehicles*
Car	66	66
Motorcycle	20	6.7
Bus	0	0
Truck	0	0
Lorry	0	0
Van	0	6
Bicycle	4	
		72.7 → 73

From D to A	*N. of vehicles*	*N. of equivalent vehicles*
Car	360	360
Motorcycle	28	9.3
Bus	0	0
Truck	4	10
Lorry	0	0
Van	0	0
Bicycle	0	
		379.3 → 379

From A to C	*N. of vehicles*	*N. of equivalent vehicles*
Car	453	453
Motorcycle	36	12
Bus	0	0
Truck	0	0
Lorry	0	0
Van	20	30
Bicycle	0	
		495

From A to D	*N. of vehicles*	*N. of equivalent vehicles*
Car	345	345
Motorcycle	60	20
Bus	0	0
Truck	0	0
Lorry	0	0
Van	16	24
Bicycle	4	
		389

From A to B	*N. of vehicles*	*N. of equivalent vehicles*
Car	508	508
Motorcycle	40	13.3
Bus	8	24
Truck	8	20
Lorry	0	0
Van	4	6
Bicycle	0	
		571.3 → 571

From B to D	*N. of vehicles*	*N. of equivalent vehicles*
Car	180	180
Motorcycle	24	8
Bus	0	0
Truck	0	0
Lorry	0	0
Van	8	12
Bicycle	16	
		200

From B to C	*N. of vehicles*	*N. of equivalent vehicles*
Car	51	51
Motorcycle	0	0
Bus	0	0
Truck	0	0
Lorry	0	0
Van	0	0
Bicycle	12	
		51

From B to A	*N. of vehicles*	*N. of equivalent vehicles*
Car	262	262
Motorcycle	20	6.7
Bus	4	12
Truck	0	0
Lorry	0	0
Van	8	12
Bicycle	8	
		292.7 → 293

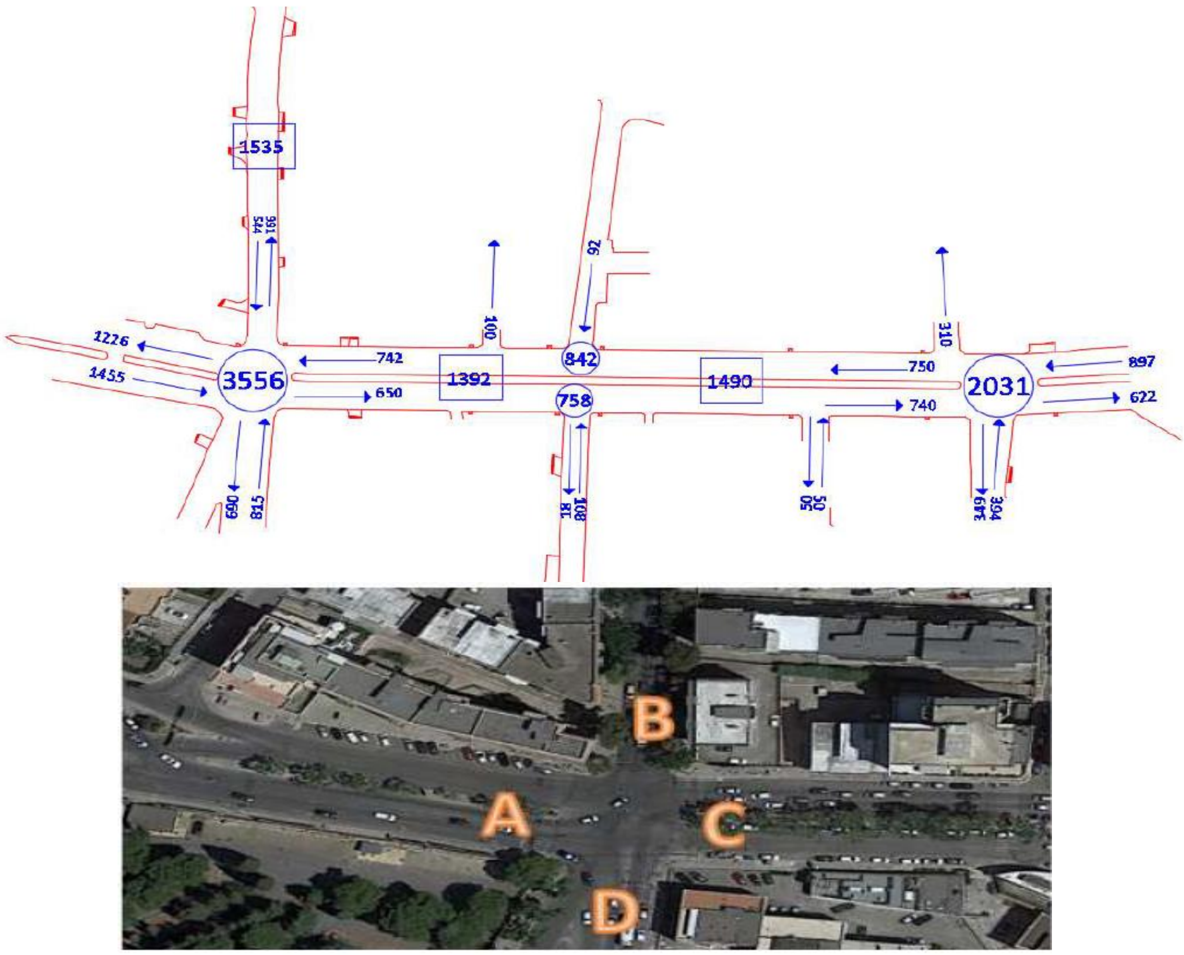

Fig. 14.44: Map of traffic flows assigned to segments and intersections (and legend of the directions used for the O/D matrix).

14.5.5 Condition diagram

The condition diagram, as indicated in the HSM manual (2010)[6], is essentially a 2D drawing representing all the details of the road network, and that highlights the site characteristics which may affect safety conditions. The purpose of this diagram is to provide information on the crash site in a way that, by overlapping this diagram with the collision diagram, it is possible to highlight problems that may have influenced the occurrence of crashes. Inside the diagram, appropriate symbols have been used in order to represent the following boundary conditions:

- private accesses and intersection with secondary roads, even if not studied in detail;
- billboards;
- street furniture;
- lighting;
- shelters for urban public transport stops;
- kiosks of commercial activities (such as newsstands, etc.);
- bins for the collection of urban waste;
- electric cabins and video surveillance systems (video cameras).
- poles (frames, curves, etc...);
- trees;
- ramps for disabled people;
- issues related to the road pavement.
- some details of the reconstruction are given below (the areas in blue on the road pavements represent bad

[6] AASHTO (2010), *Highway Safety Manual, First Edition*, Transportation Research Board, National Research Council, Washington D.C., USA.

pavement conditions such as local cracks or area-wide problems).

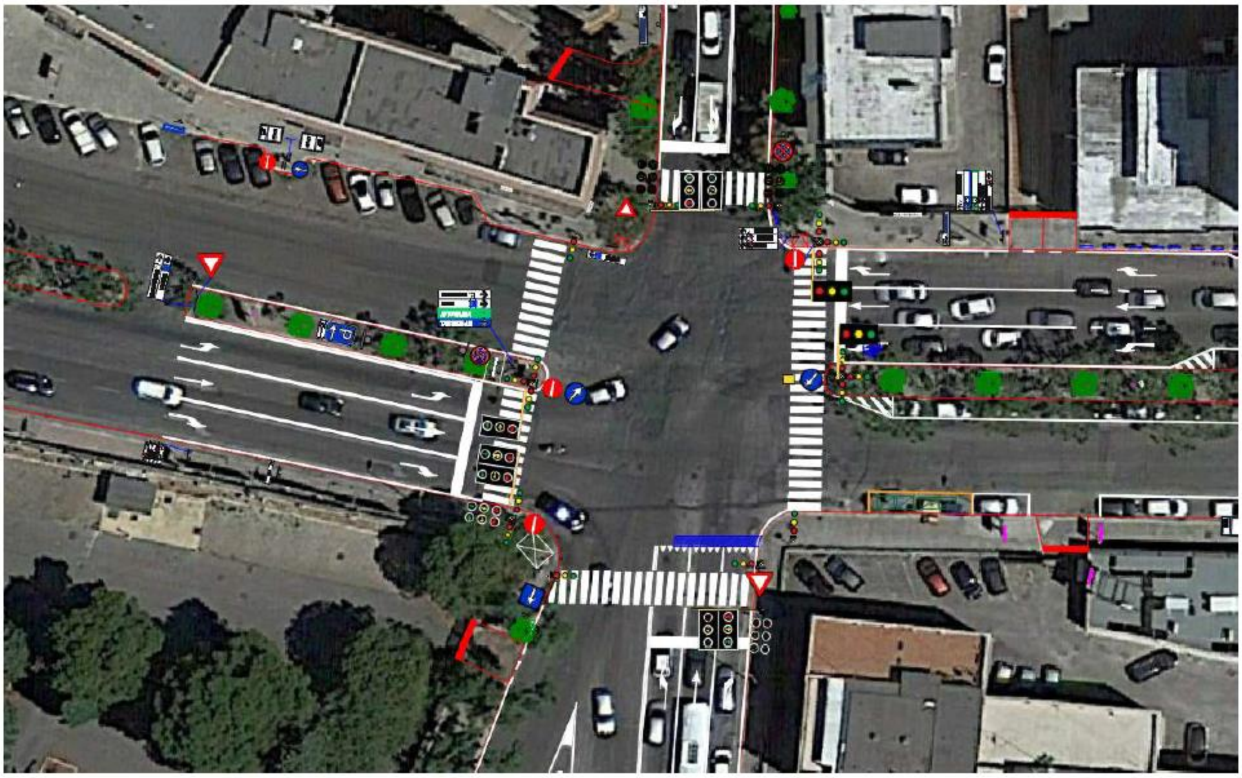

Fig. 14.45: Detail of the condition diagram - Intersection I (photo source Google Earth).

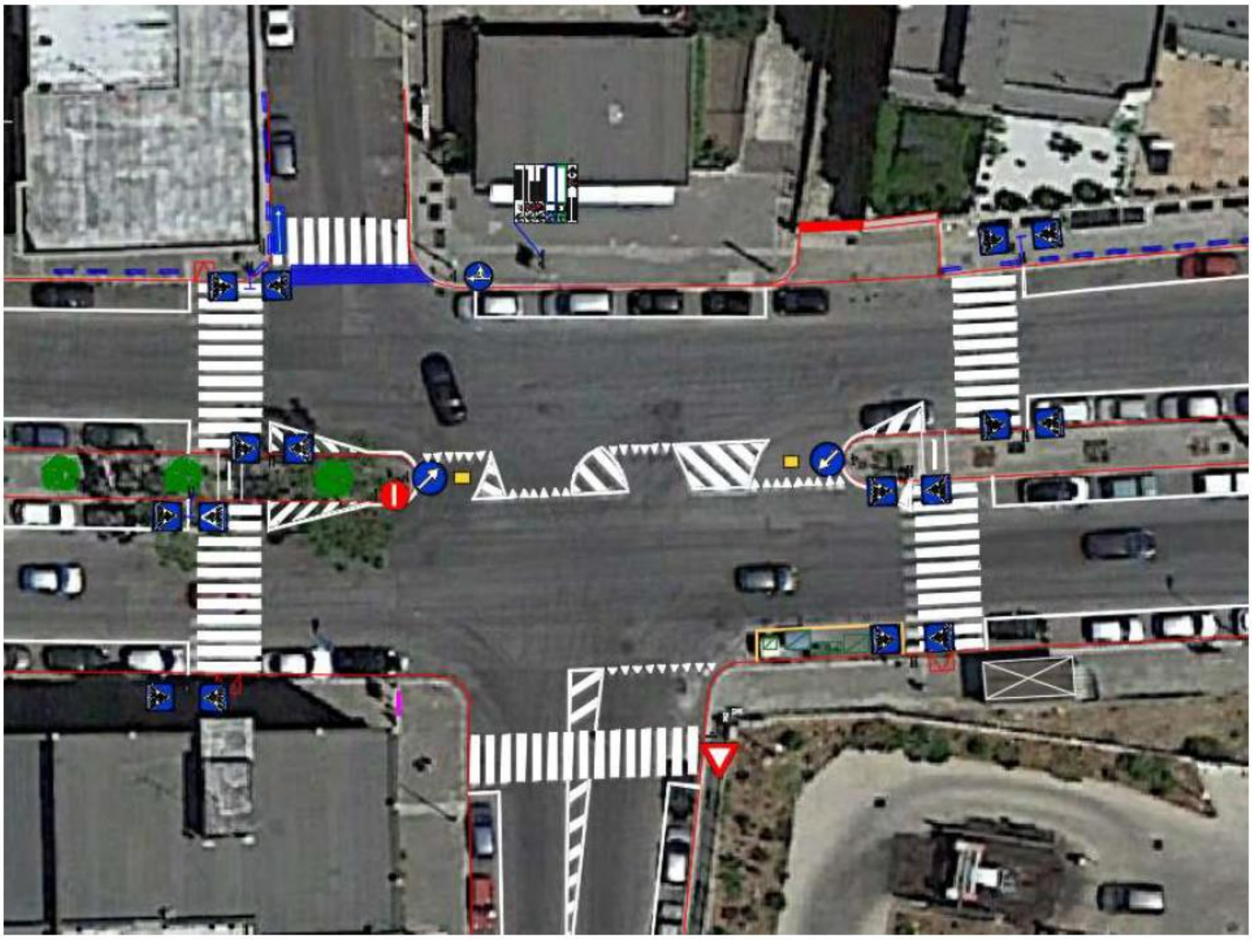

Fig. 14.46: Detail of the condition diagram - Intersection IV (photo source Google Earth).

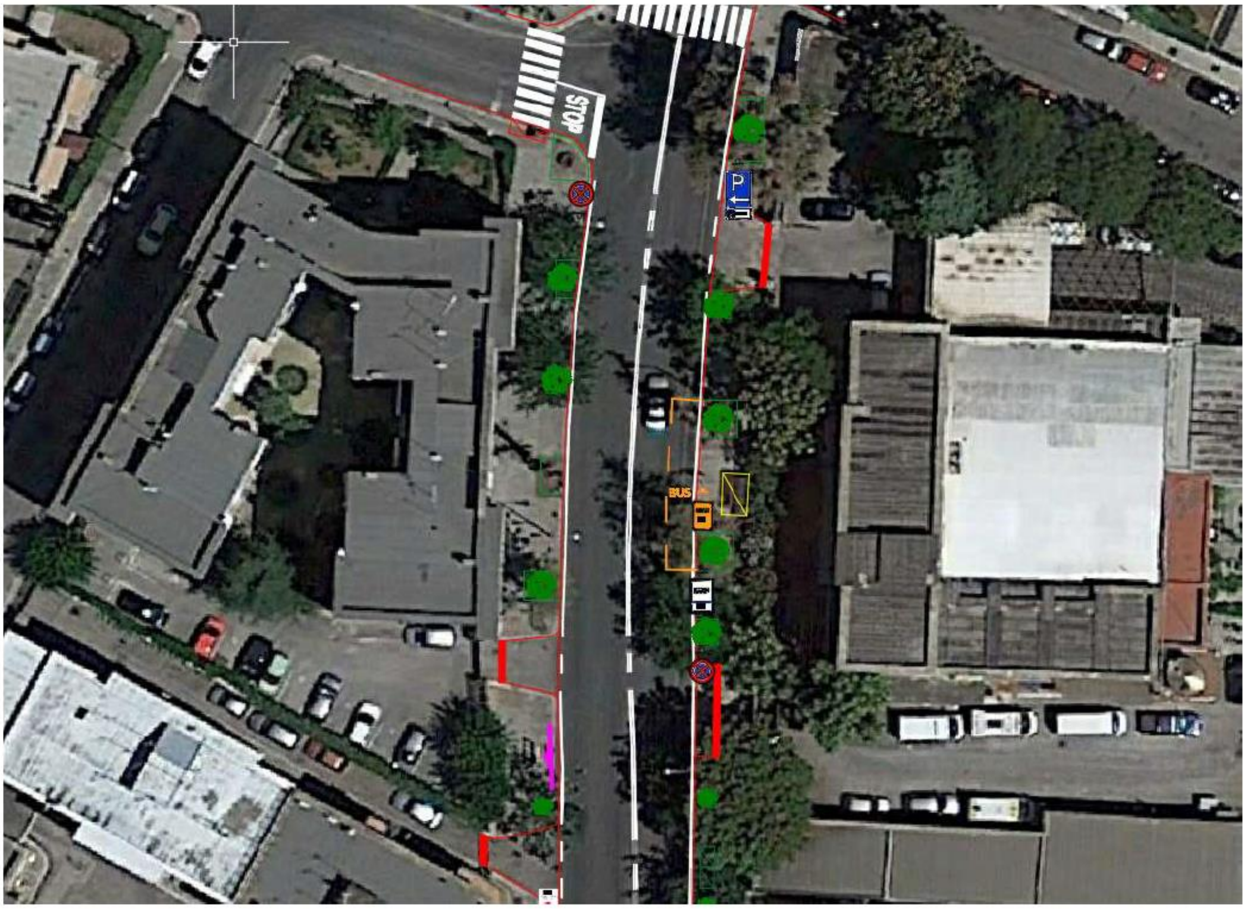

Fig. 14.47: Detail of the condition diagram - Viale Orazio Flacco (photo source Google Earth).

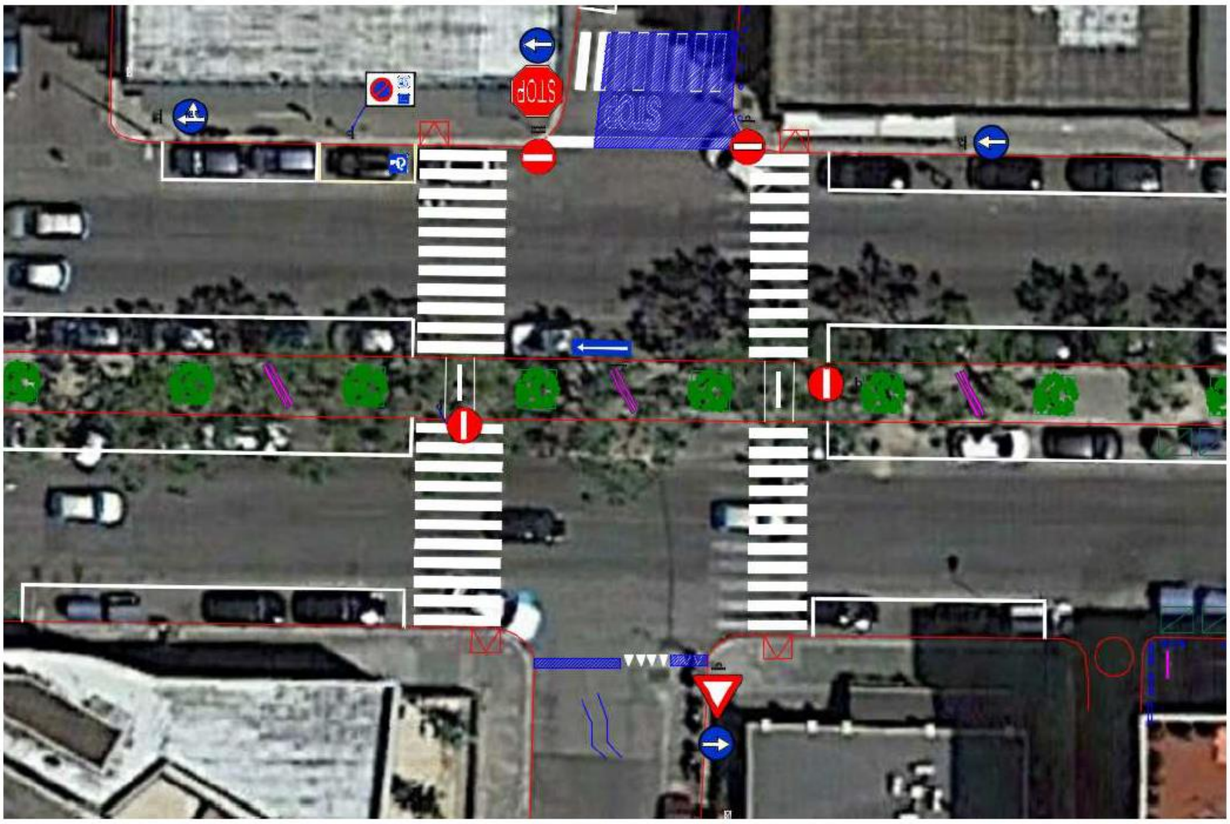

Fig. 14.48: Detail of the condition diagram - Intersections II and III (photo source Google Earth).

14.5.6 Collision diagram

Based on the location of crashes (defined in paragraph 14.9.3), evaluating the data present in table 14.17, which contains relevant information for each crash that occurred in the area of study, and using the information from the condition diagram, the collision diagram was defined. The first location of crashes, in some cases, was not actually in tune with all the data analysed, so some crashes were appropriately relocated.

For each type of crash, the symbolism suggested by the Highway Safety Manual (2010)[6] and shown in Figure 14.49, was used. Finally, the direction of the arrows defines the presumed direction of travel of the vehicles.

Fig. 14.49: Symbols used in the Collision Diagram (as based on the HSM).

For each crash, the following characteristics were added on the map:

- crash identification number;
- season: E (summer), A (autumn), I (winter), P (spring);
- day: FR (weekday), FS (holiday), WE (Weekend);
- period: M (morning), PM (afternoon), S (evening), N (night);
- weather: S (clear), PG (rainy), NV (snowy), NB (foggy), V (windy), AT (other);
- pavement conditions: A (dry), B (wet), G (icy).

These pieces of information have been reported in the collision diagram as follows:

- upper row: Season - Day - Period (e.g.: E - FR - M);
- lower row: Weather - pavement (e.g.: S - A).

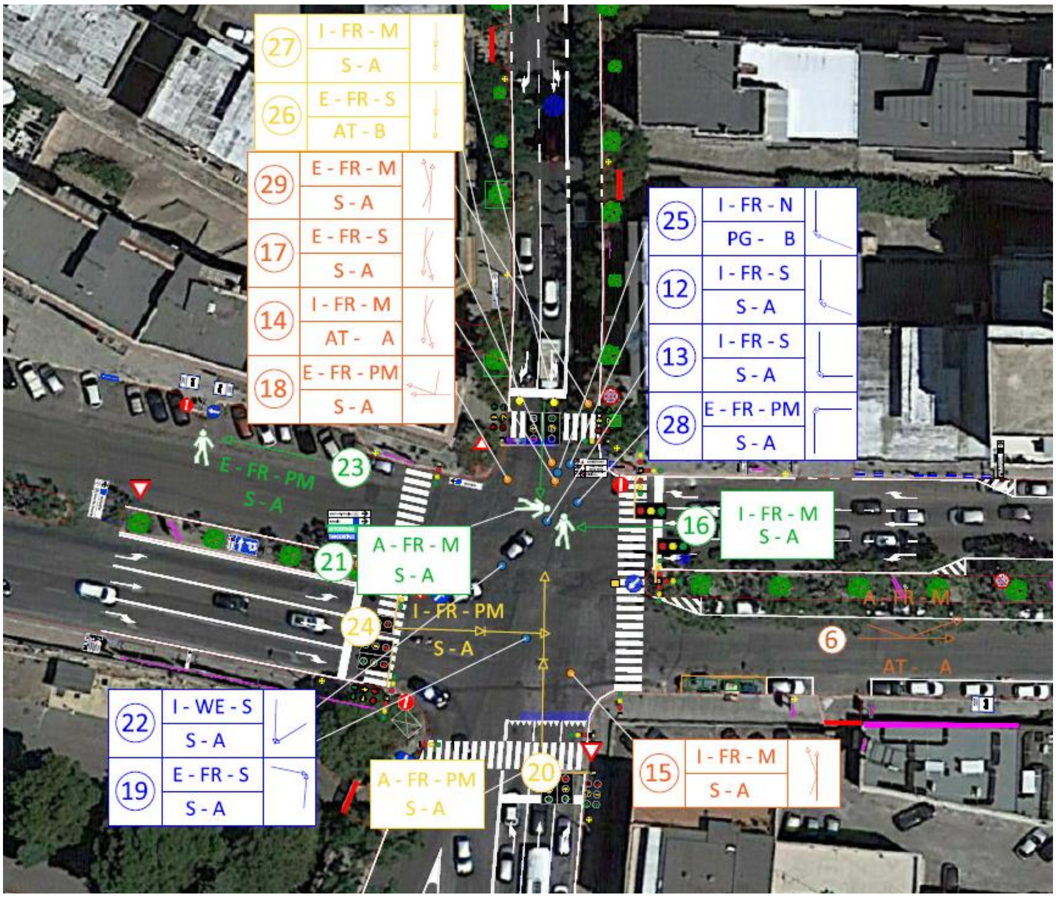

Fig. 14.50: Detailed Collision Diagram - Intersection I (photo source Google Earth).

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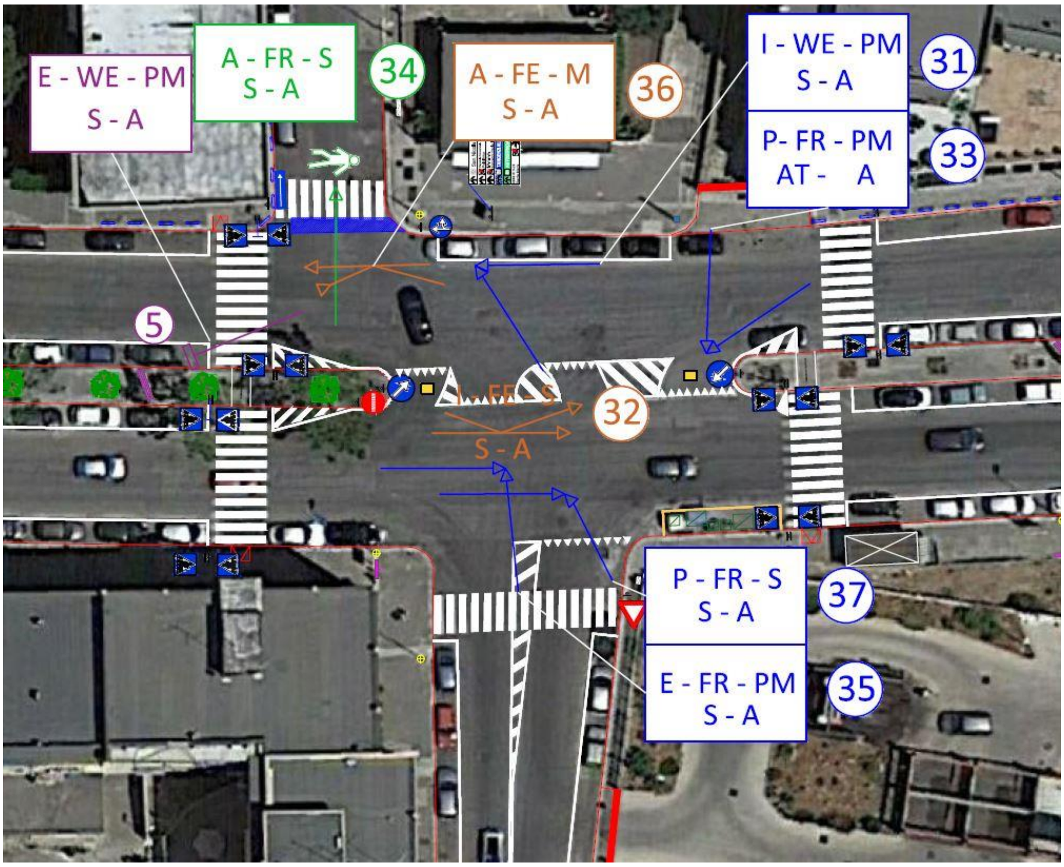

Fig. 14.51: Detailed Collision Diagram - Intersection IV (photo source Google Earth).

14.6 Modeling and application of the EB predictive method

14.6.1 Predictive method (Highway Safety Manual, HSM, 2010[6])

It is necessary to estimate safety performances both before and after the implementation of countermeasures. This is done with the aim of quantifying the effective safety improvement of the analysed infrastructure after the implementation of countermeasures on the whole road or at specific points.

The difference between the crashes before and after the implementation of the countermeasures is a metric of the introduced benefit. The crash estimation method applied in this case study is the one provided by the Highway Safety Manual (2010)[6], the EB (Empirical Bayesian) predictive method. The output of this method is the Expected mean crash frequency $N_{Expected}$, i.e., the estimation of the annual crash number which will occur for a fixed number of years after the moment of the analysis. This output is obtained by combining the Observed mean crash frequency $N_{Observed}$ at a given site (so the actual number of crashes occurred over the years of analysis) and the Predicted mean crash frequency $N_{Predicted}$ (which is the annual number of crashes predicted for the site) through the application of a regression method:

$$N_{Expected} = (N_{Predicted}\, x\, w) + (1 - w)N_{Observed} \qquad \text{(Eq. 14-7)}$$

Where:

- $w = \frac{1}{1+k\ \sum_{years\ of\ analysis} N_{Predicted}}$
- k = overdispersion parameter.

The idea of using the combination of these two frequencies rises from the need of compensating the statistical errors of both the parameters:

- the statistical reliability of the Observed frequency requires long time windows of analysis, since the crash occurrence is a random and rare event, using means coming from short time windows of analysis could

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lead to biased estimations possibly characterized by the RTM (regression to the mean) bias;

- the Predicted frequency may suffer from errors due to changes in road conditions (which are likely to happen after a given period), since it has been calculated through a model applied to a specific site with given geometric characteristics and traffic volumes.

The Predicted Mean frequency is calculated through the Safety Performance Functions (SPFs), which relate the annual average daily traffic (vehicle exposure to crashes) to the crash frequency for a specific site. However, the application of the HSM SPFs was not followed. In fact, the HSM provides SPFs for *Arterials* which are transit roads in urban and suburban contexts, with geometric characteristics and speed limits not comparable to the ones analysed in this study. Hence, different approaches have been used for the Predicted Mean frequency calculation. The research for an ad-hoc approach has been deeply influenced by the statistical reliability and overlapping with the HSM methodology, but also by the geographical and cultural context of the analysed sites.

It is crucial, in an urban context too, considering the human behaviour. Hence, European models were preferred to American ones since the European cultural and urban infrastructure background was deemed as closer to the Italian one (considering also the previously mentioned issue of the urban HSM SPF). Albeit the EB-empirical method enables the calculation of the severity distribution of expected crashes, in the analysed case, the expected mean crash frequency has been calculated only for crashes with deaths and injuries. In fact, the observed crashes are only fatal + injury crashes and the employed models have been calibrated starting from database of fatal + injury crashes. Thus, the total crashes can be calculated from the fatal + injury frequency after the application of coefficients calibrated for the Italian case.

In particular, the estimates of costs from traffic crashes in European countries found in Wijnen et al. (2017)[7], were used. The relevant data for Italy (year 2015) retrieved from this report are listed below:

- fatal crashes (K) = 3,847
- severe injuries crashes (resulting in hospital cares for more than 24 hours) (A/B) = 29,724
- slight injuries crashes (C) = 177,833
- property damage only crashes (O) = 1,559,185

Assuming that the 2015 is a close enough year to ensure an acceptable statistical data fluctuation, it was possible to calculate the percentage of fatal + injury crashes out of the total number of crashes:

$$\%crashes_{KABC\ Italy,2015} = \frac{3847+29724+177833}{3847+29724+177833+1559185} 100 \cong 12\% \quad \text{(Eq. 14-8)}$$

This percentage will allow to calculate the crashes of all the severities occurred on the analysed network and expected by the models. The "possible injuries" C crashes, even if they are difficult to be detected, were considered as *slight injury crashes*.

14.6.2 Crash Modification Factors (CMFs)

The frequency, calculated through a baseline SPF (the SPFs used in this example are shown in the following paragraph), must be modified by coefficients (Crash Modification Factors, CMFs) to consider both: a) the actual site conditions which may be different from the ones for which the model was estimated and b) the effects of countermeasures on the crash frequency. The value of $N_{Predicted}$ corrected by the CMFs, is reported as follows:

$$N_{Predicted\ CMF} = N_{Predicted}\ x\ (CMF1\ x\ CMF2\ x\ CMF3 \ldots\ x\ CMFn)\ x\ Cc \quad \text{(Eq. 14-9)}$$

Where:

- $N_{Predicted}$ is the predicted mean crash frequency for a specific year for the site of interest;
- $N_{Predicted\ CMF}$, is the predicted mean crash frequency for a specific year for the site of interest, modified by the CMFs, Crash Modification Factors;
- CMF_1, CMF_2, CMF_3, …CMFn; are the Crash Modification Factors, which take into account the variations from the baseline conditions;
- Cc is the calibration coefficient that considers local conditions. Studies about the calibration coefficient for crashes in Italian urban areas were not found, so Cc was set equal to 1 for both the models used in this preliminary application example.

[7] Wijnen W., Weijermars W., Vanden Berghe W., Schoeters A., Bauer R., Carnis L., Elvik R., Theofilatos A., Filtness A., Reed S., Perez C., Martensen H., (2017) *Crash cost estimates for European countries, Deliverable 3.2*, of the H2020 project SafetyCube.

Besides the variables considered in the selected SPFs for urban road segments and intersections, several other additional CMFs were retrieved from the database on the website www.cmfclearinghouse.org. From this source, it is possible to assess the reliability of the CMFs by means of the rating provided (star rating from 0 to 5, 5 is a good quality CMF). Other CMFs were mainly taken from the *Reference Guide of Federal Highway Administration*[8] and the Italian *CEREMSS*[9]. The CMF which accounts for different visibility conditions (in terms of sight distance) at the intersections, was taken from the recent document published by the NASEM (2018)[10].

However, it should be highlighted that the CMFs used for estimating the expected crash frequency in the *current conditions* (as built) take into account all the details of the road segment or the intersection which were not modelled through the reference SPF. The CMFs used for estimating the expected crash frequency in the *design conditions* evaluate the impacts of the implemented countermeasures on the expected crashes, regardless of the CMFs already considered in the current conditions.

CMFs were used for both the current and design conditions in order to increase the overlapping between the studied models and the actual case of Bari.

The CMFs retrieved from different sources were generally referred to "all" vehicles crash types and severities. Instead, when the CMFs are related to some specific kinds of vehicles, users or crash types, they were weighted for the observed number of crashes occurred to those categories.

Different CMFs related to the same category (e.g., in case of crosswalks, there could be different CMFs for horizontal and vertical traffic signs) were combined. However, the combination of CMFs for a single intervention was avoided as much as possible. This was made considering the lowest value of the CMF among the CMFs for the same intervention. This solution attempts at accounting for all the positive effects of countermeasures, without making errors due to the overlapping effects of the same countermeasures.

14.6.3 Safety Performance Functions for urban road segments

The SPF developed by Greibe, 2003[11] was used to calculate the predicted mean crash frequency for segments. The model equation is the following:

$$E(\mu) = a\, AADT^{p} \exp\left(\sum \beta_j x_{ij}\right) \qquad \left[\frac{crashes}{year\ x\ km}\right] \qquad \text{(Eq. 14-10)}$$

Where:

- $E(\mu)$ is the expected mean crash frequency;
- AADT is the Annual Average Daily Traffic volume;
- x_{ij} are the descriptive variables of the road geometry and urban environment where the road segment is placed;
- a, p and β_j are the estimated parameters of the model.

The statistically significant variables included in the cited model were:

- *land use*: the type and function of the buildings on the road sides have a strong impact on crashes, the lower is the building density the lower is the crash risk;
- *speed limits:* roads with higher speed limits are associated to a lower crash risk, probably because low-speed roads are in typically urban areas, where there are several interactions with vulnerable users (like pedestrians, cyclists and motorcyclists);
- *road width:* medium-large lanes reduce the crash risk;
- *on-street parking spots:* on-street parking spots increase the crash risk because of the number of conflict points between parking vehicles and travelling vehicles as well as between pedestrians and vehicles;
- *number of accesses/km:* roads with no accesses or with a relevant number of accesses (residential neighbourhood roads) show a low crash risk;
- *number of intersections with minor roads:* the higher is the number of intersections, the greater is the crash risk.

[8] U.S. Department of Transportation Federal Highway Administration, FHWA (2008), *Desktop reference for crash reduction factors*, Report no. FHWA-SA-08-011, September.

[9] Ceremss, Lazio: https://ceremsslazio.astralspa.it/ceremss/

[10] National Academies of Sciences, Engineering, and Medicine (2018), *Guidance for Evaluating the Safety Impacts of Intersection Sight Distance,* The National Academies Press., Washington, D.C., USA.

[11] Greibe P. (2003), "Accident prediction models for urban roads", *Accident Analysis & Prevention*, 35(2), 273-285.

A Poisson probability distribution function was used by Greibe (2003)[11] and then the overdispersion coefficient "k" is not available. Hence, in order to adapt this model in the EB method, the weight "w" was hypothesized.

A spreadsheet based on this formulation was used for the calculation of the expected mean crash frequency. In the spreadsheet, all the model variables are transformed into CMFs, considering the variation of crash frequencies with respect to the baseline value of the variable. The number of observed crashes on each homogenous segment should be manually inserted into the spreadsheet in order to obtain EB estimates.

The spreadsheet includes two different models:

- *current conditions:* the reported values are related to the current conditions (as-built, before the design starts) of the segment. The CMFs are calculated for both the variables in the Greibe's model (conditions different than the model baseline conditions assumed) and for the conditions to be modelled which were not considered in the Greibe's model.
- *design conditions:* the values related to the design hypothesis are reported. The CMFs consider the effect of the adopted countermeasures on crashes (besides the modifications in the Greibe's model variables, which are automatically updated in the spreadsheet, according to the considered values).

$$Nexp_{design} = Nexp_{current} \frac{\prod CMF{,}design}{\prod CMF{,}current} \frac{Nspf\ (only\ AADT)_{design}}{Nspf\ (only\ AADT)_{current}} \qquad \text{(Eq. 14-11)}$$

The CMF_{design} is the product of all the CMFs considering the countermeasures, the $CMF_{current}$ is the product of all the CMFs considering the actual site characteristics. The ratio between the two products of CMFs is multiplied by the expected mean crash frequency (current conditions) and by the ratio of the design N_{SPF} by the current N_{SPF} (to account for differences in the AADT values at the end of the countermeasures lifetime).

The difference between the two expected crash values (current and design) indicates the safety improvement due to the adopted countermeasures.

Some remarks:

- the β_j (accesses) values provided for the ranges (0 - 5), (> 40), (5 - 40) in the reference study are not entirely coherent with the actual observed effects of accesses on safety in the case study analyzed. However, the β value for the range (5- 40) accesses is 1.43, the most dangerous;
- in an urban analysis, the procedure of splitting the road segments into homogenous parts could not rely on significant vertical-horizontal alignment variations and cross section variations. The division into homogeneous parts was applied when the values of the Greibe's model changed, to have homogenous values of the Greibe's model variables on each part;
- each variable is associated to a unique value of the parameter β_j. Some exceptions could be the land use and the parking conditions because they could vary on the two sides of the same cross road sections. In these cases, the β_j is a mean between the two conditions, and 0.5 has been set as the value to be used in case of two variables referring to the same macro-item.

In some cases, the variables may be different even within the same homogeneous segment (e.g., parking and land use between the left and right roadsides), thus the mean value between the different conditions is considered.

14.6.4 Safety Performance Functions for urban intersections

The study about intersections was run taking into account the model developed by Gomes et al. (2012)[1], because it is more recent than the Danish model and closer to the Italian conditions.

Another important aspect is that Lisbon can be deemed as more similar to Bari for climatic and geographic characteristics, so the model could be more suitable for the Italian reality. This model was developed by using a negative binomial distribution function and then the overdispersion parameter is available. In order to isolate the crashes occurred at intersections only, Gomes et al. (2012)[1] have considered the crashes happened within a length of 40 m from the centre of the intersection. This length was the theorical radius of a circular area. This procedure was used in this application example too.

The model equation is the following:

$$\mu_{it} = \beta_0\ x\ (F_{1it} + F_{2it})^{\beta_1}\ x\ (\frac{F_{2it}}{F_{1it}+F_{2it}})^{\beta_2}\ x\ e^{\sum_{k=3}^{n}\beta_k x_k} \quad [\frac{crashes}{year}] \qquad \text{(Eq. 14-12)}$$

where:

- μ_{it} is the predicted mean crash frequency for the given intersection *i*;
- F_{1it} is the AADT of the main road, 1, approaching to the intersection *i*;
- F_{2it} is the AADT of the secondary road, 2, approaching to the intersection *i*;
- x_k is a generic predictor;
- β_k is the estimated coefficient, associated to each predictor.

Gomes et al. (2012)[1] used two different models, one for the 3-legged intersections and the other for the 4-legged intersections. These two models have different predictors, reported as follows:

- 3-legged intersections;
    - *LB*: Lane balance, a binary parameter whose value is 1 if the secondary road and primary road approaching to the intersection have the same number of lanes, 0 if not;
    - *MMAJ1*: binary parameter which considers the presence (1) or not (0) of the median in one segment of the main road;
    - *MMAJ2*: binary parameter which considers the presence (1) or not (0) of the median in both the approaching segments of the main road;
    - *MMIN*: binary parameter which considers the presence (1) or not (0) of the median in both the approaching segments of the secondary road;
    - *RTPMAJ*: binary parameter which considers the presence (1) or not (0) of the specialized lane for the right turning manoeuvre;
- 4-legged intersections;
    - *LB*: Lane balance, a binary parameter whose value is 1 if the secondary road and primary road approaching to the intersection have the same number of lanes, 0 if not;
    - *LMAJT7*: it is a binary parameter which gives an indication about the number of road lanes: if they are more than 3, the parameter is 1, otherwise 0;
    - *LWMIN*: it is the average width of the secondary road lane (numerical value);
    - *RTPMAJ*: binary parameter which considers the presence (1) or not (0) of the specialized lane for the right turning manoeuvre;
    - *LOW*: it identifies the number of one-way road segments (numerical value).

Some additional remarks are reported as follows:

- two different models are provided in the spreadsheet, one for the current and the other for the design conditions, as well as for the segments;
- traffic lights are not included in the model; thus, they are considered through CMFs.

14.7 Selection of countermeasures

This section describes the countermeasures proposed for the road intersections and the road segments (divided in homogeneous sub-segments) aiming at increasing the safety of the analysed network. Two distinct sets of countermeasures are discussed:

- short-term countermeasures.
- long-term countermeasures.

The short-term countermeasures include interventions that do not require road geometric changes; while long-term countermeasures require significant changes to the current road environment, possibly including some new road elements or cycle paths/pedestrian areas.

Moreover, ordinary maintenance to restore the road functionality was added to countermeasures, if relevant.

14.7.1 Possible countermeasures

In the following, countermeasures for each road segment (divided in homogeneous parts) are presented, without considering the intersection area (delimited by the 40 metres distance from the centre of the intersections: Gomes et al., 2012[1]). They are graphically superimposed on the initial drawings shown in the previous sections.

Note that they are based on the problems highlighted in the diagnosis stage and the crash types and circumstances, as reconstructed and interpreted from the available database (some of the reconstructions may have been hypothesized because of the partially insufficient data).

14.7.1.1 Segment: "Viale Papa Giovanni XXIII, T1 – Northern part"

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, this homogenous part of Viale Papa Giovanni XXIII shows the following problems:

- there are 2 driveways, even without consequences for road safety in terms of driveway-related crashes;
- the crosswalk at the intersection with Via Papa Bonifacio IX is not provided with the appropriate vertical traffic-sign;
- the median width has several billboards on it;
- there are slight pavement cracks at the beginning of the specialized right turning lane.

Tab. 14.22: Characteristics of the segment: "Viale Papa Giovanni XXIII, T1 - Northern part" (model variables such as AADT, segment length, specific geometric elements and observed crashes).

Model Variables	*Homogenous part 1*
AADT	8420
Segment length [km]	0.036
Horizontal alignment element	Tangent
Number of crashes	0

Fig. 14.52: View of the segment: "Viale Papa Giovanni XXIII, T1 - Northern part" (photo source Google Earth).

Two sets of countermeasures have then been proposed for the short-term interventions:

- set A - Short-term – Speed:
  - transverse optical speed bars, to reduce the vehicle speeds during their approach to the intersection.
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - high visibility crosswalks, split in two parts (two-steps crossing) with a pedestrian refuge island and vertical traffic-signs for the crosswalks;
  - new crosswalk at the intersection with Via Papa Bonifacio IX;
  - installation of barriers on the sidewalks to avoid irregular crossing manoeuvres by pedestrians;
  - new road markings to divide lanes;
  - new zebra patterns with rubber curbs to prevent irregular parking in case of driveways.

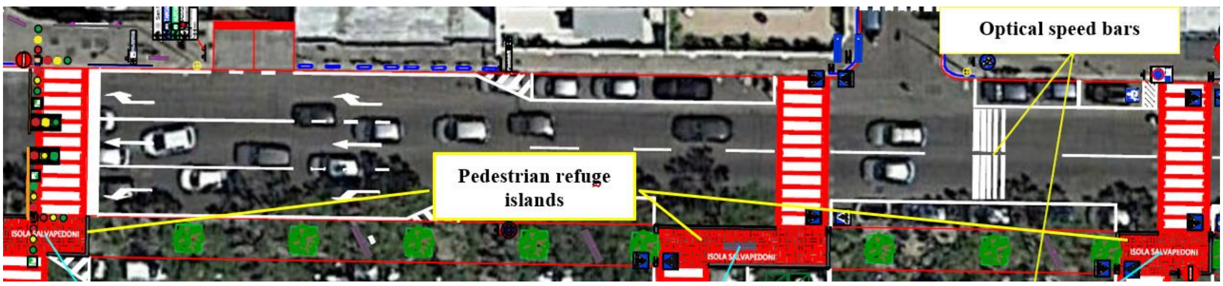

Fig. 14.53: Short-term countermeasures T1 - Northern part (transverse optical speed bar and pedestrian refuge islands).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

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The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- implementation of new cycle paths (bi-directional), by replacing the traffic island;
- new cycle/pedestrian crossings with adequate related traffic signs;
- installation of barriers to protect the cycle path;
- forbid vehicles stopping where there is the cycle path.

Fig. 14.54: Long-term countermeasures T1 - Northern part (Sosta proibita in prossimità della pista ciclabile = prohibited parking next to the cycle lane).

14.7.1.2 Segment: "Viale Papa Giovanni XXIII, T2 – Northern part"

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, this homogenous part of Viale Papa Giovanni XXIII shows the following problems:

- 2 crashes occurred;
- the 2 crashes are: 1) pedestrian collision (due to illegal pedestrian crossing), 2) rear-end caused by distracted driving;
- the median width has several billboards on it;
- there are slight pavement cracks.

Tab. 14.23: Characteristics of the segment: "Viale Papa Giovanni XXIII, T2 - Northern part" (model variables such as AADT, segment length, specific geometric element and observed crashes).

Model Variables	*Homogenous part 1*
AADT	7500
Segment length [km]	0.068
Horizontal alignment element	Tangent
Number of crashes	2

Fig. 14.55: View of the segment: "Viale Papa Giovanni XXIII, T2 - Northern part" (photo source Google Earth).

Two sets of countermeasures have then been proposed for the short-term interventions:

- set A - Short-term – Speed:
  - transverse optical speed bars, to reduce the vehicle speeds during their approach to the intersection.
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - high visibility crosswalks, split in two parts (two-steps crossing) with a pedestrian refuge island and vertical traffic-signs for the crosswalks;

- installation of barriers on the sidewalks to avoid irregular crossing manoeuvres by pedestrians;
- new road markings to divide lanes;
- new zebra patterns with rubber curbs to prevent irregular parking in case of driveways.

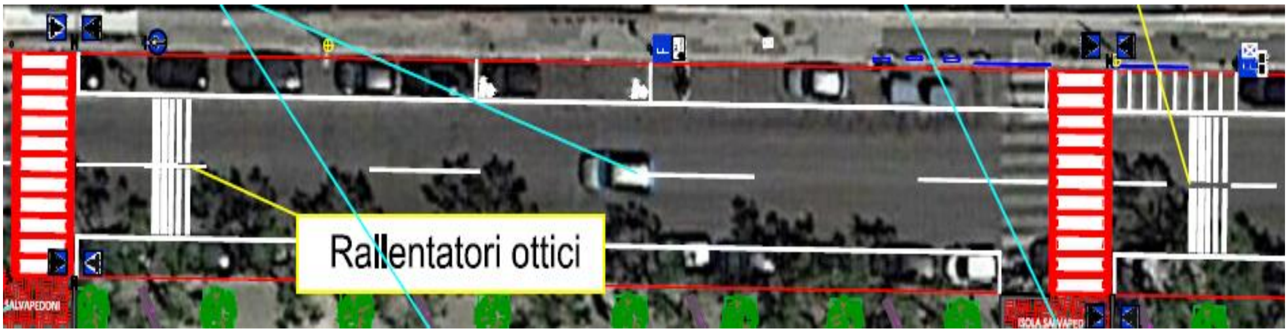

Fig. 14.56: Short-term Countermeasuers T2 - Northern part (Rallentatori ottici= transverse optical speed bars).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- implementation of new cycle paths (bi-directional), by replacing the traffic island;
- new cycle/pedestrian crossings with adequate related traffic signs;
- installation of barriers to protect the cycle path;
- forbid vehicles stopping where there is the cycle path.

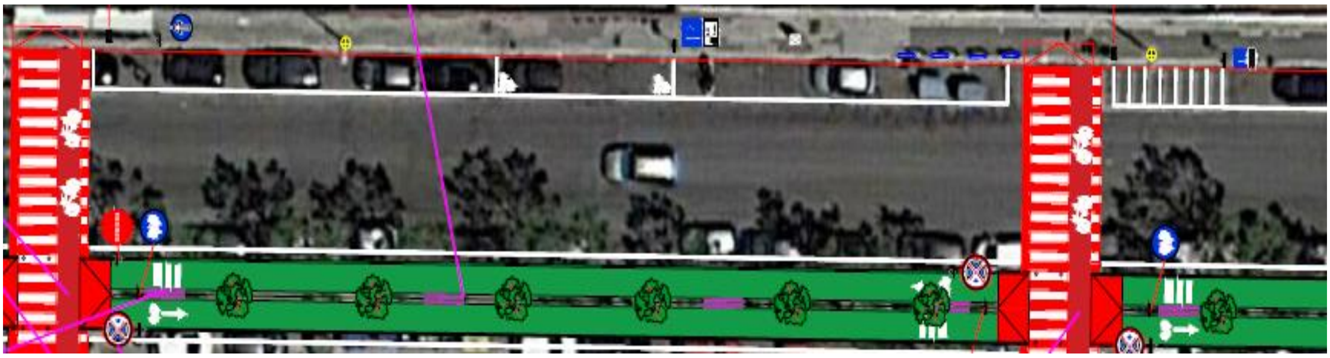

Fig. 14.57: Long-term Countermeasures T2 - Northern part.

14.7.1.3 Segment: "Viale Papa Giovanni XXIII, T1 -Southern part"

Tab. 14.24: Characteristics of the segment: "Viale Papa Giovanni XXIII, T1 - Southern part" (model variables such as AADT, segment length, specific geometric element and observed crashes).

Model Variables	*Homogenous part 1*
AADT	6500
Segment length [km]	0.035
Horizontal alignment element	Tangent
Number of crashes	1

Fig. 14.58: View of the segment: "Viale Papa Giovanni XXIII, T1 - Southern part" (photo source Google Earth).

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, this homogenous part of Viale Papa Giovanni XXIII shows the following problems:

- 1 crash occurred;
- the crash was a side impact collision due to high speeds;
- the median width has several billboards on it;
- there are slight pavement cracks.

Two sets of countermeasures have been proposed for the short-term interventions:

- set A - Short-term – Speed:
  - transverse optical speed bars, to reduce the vehicle speeds during their approach to the intersection.
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - high visibility crosswalks, split in two parts (two-steps crossing) with a pedestrian refuge island and vertical traffic-signs for the crosswalks;
  - new crosswalk at the intersection with Via Papa Bonifacio IX;
  - installation of barriers on the sidewalks to avoid irregular crossing manoeuvres by pedestrians;
  - new road markings to divide lanes;
  - new zebra patterns with rubber curbs to prevent irregular parking in case of driveways.

Fig. 14.59: Short-term countermeasures T1 - Southern part (pedestrian refuge islands).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- implementation of new cycle paths (bi-directional), by replacing the traffic island;
- new cycle/pedestrian crossings with adequate related traffic signs;
- installation of barriers to protect the cycle path;
- forbid vehicles stopping where there is the cycle path.

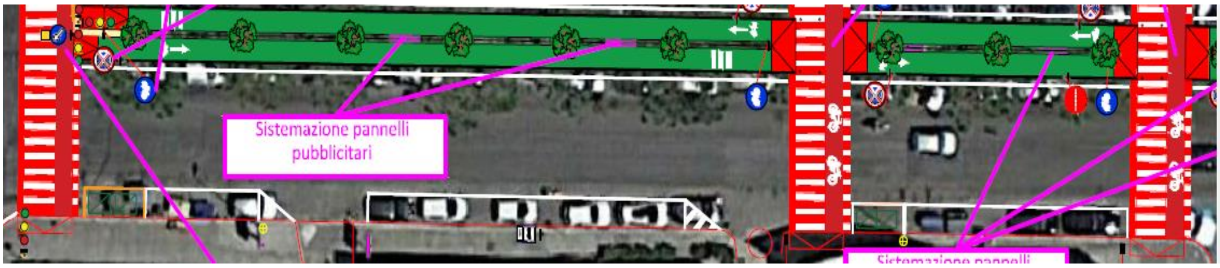

Fig. 14.60: Long-term countermeasures T1 - Southern part (sistemazione pannelli pubblicitari = adjustment of the advertising panels' location).

14.7.1.4 Segment: "Viale Papa Giovanni XXIII, T2 - Southern part"

Tab. 14.25: Characteristics of the segment: "Viale Papa Giovanni XXIII, T2 - Southern part" (model variables such as AADT, segment length, specific geometric element and observed crashes).

Model Variables	*Homogenous part 1*
AADT	7400
Segment length [km]	0.068
Horizontal alignment element	Tangent
Number of crashes	2

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Fig. 14.61: View of the segment: "Viale Papa Giovanni XXIII, T2 - Southern part" (photo source Google Earth).

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, this homogenous part of Viale Papa Giovanni XXIII shows the following problems:

- in this part, 2 crashes occurred;
- the crashes were: an angle collision (number 7) related to a driveway having inadequate sight distance and a rear-end (number 3) due to inadequate safe headways;
- the median width has several billboards on it;
- there are slight pavement cracks.

Two sets have been proposed for the short-term interventions:

- set A - Short-term – Speed:
  - transverse optical speed bars, to reduce the vehicle speeds during their approach to the intersection.
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - high visibility crosswalks, split in two parts (two-steps crossing) with a pedestrian refuge island and vertical traffic-signs for the crosswalks;
  - new crosswalk at the intersection with Via Papa Bonifacio IX;
  - installation of barriers on the sidewalks to avoid irregular crossing manoeuvres by pedestrians;
  - new road markings to divide lanes;
  - new zebra patterns with rubber curbs to prevent irregular parking in case of driveways.

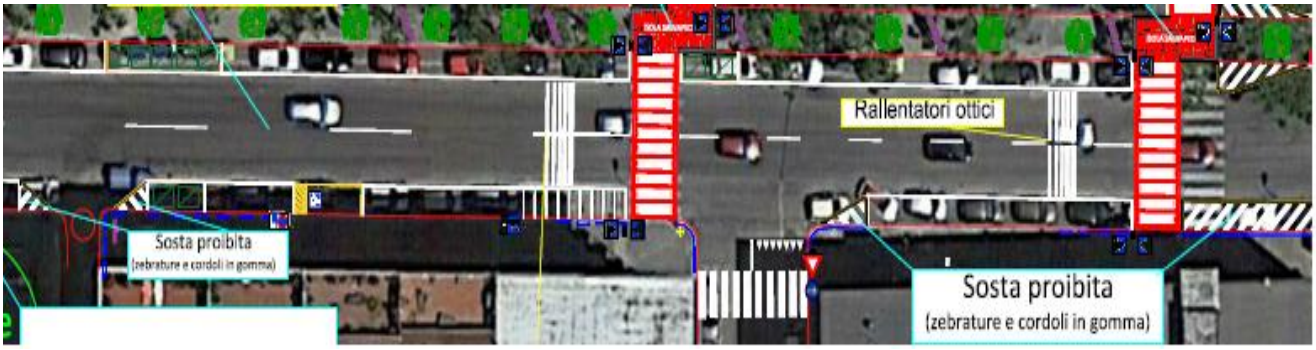

Fig. 14.62: Short-term countermeasures T2 - Southern part" (Sosta proibita = prohibited parking/zebrature e cordoli in gomma = Zebra patterns and rubber kerbs).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- implementation of new cycle paths (bi-directional), by replacing the traffic island;
- new cycle/pedestrian crossings with adequate related traffic signs;
- installation of barriers to protect the cycle path;
- forbid vehicles stopping where there is the cycle path.

Fig. 14.63: Long-term countermeasures T2 - Southern part" (Sosta proibita in prossimità della pista ciclabile = prohibited parking close to cycle lane/realizzazione di rampe per pista ciclabile = construction of cycle path ramps).

14.7.1.5 Segment: "Viale Orazio Flacco"

Tab. 14.26: Characteristics of the segment: "Viale Orazio Flacco" (model variables such as AADT, segment length, specific geometric element and observed crashes).

Model Variables	*Homogenous part 1*
AADT	15350
Segment length [km]	0.113
Horizontal alignment element	Tangent + curve
Number of crashes	2

Fig. 14.64: View of the segment: "Viale Orazio Flacco" (photo source Google Earth).

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, the segment Viale Orazio Flacco shows the following problems:

- 2 crashes occurred;
- there are slight pavement cracks where the arrow markings start;
- trees and vegetation are present on the sidewalk, covering give-way signs close to traffic lights;
- the bus stop is not provided with crosswalks to allow passengers to cross the street;
- the two crashes were: an angle collision (number 8) due to ignoring the stop sign on Via Storelli and a rear-end (number 9) related to the same secondary road, Via Storelli.

Two sets of countermeasures have been proposed for the short-term interventions:

- set A - Short-term – Speed:
  - transverse optical speed bars, to reduce the vehicle speeds during their approach to the intersection.
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - high visibility crosswalks, at the bus stop, with vertical traffic signs provided with LEDs;
  - pruning the vegetation on the roadside and install vertical danger traffic signs with integrative panels "trees on the roadside"
  - installation of vertical traffic signs to warn drivers about the different lane directions.

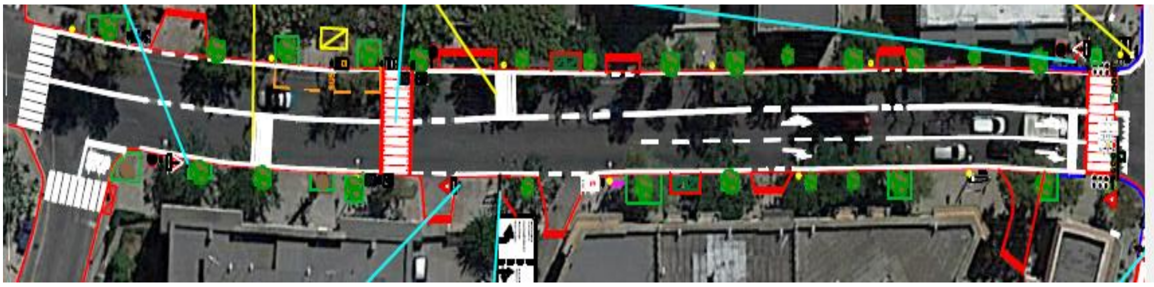

Fig. 14.65: Short-Term countermeasures Viale Orazio Flacco.

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

Long-term countermeasures (set C) similar to those reported for the previous segments are not considered for this segment, mainly because cycle paths on it are not expected from urban mobility plans.

Moreover, the removal of trees on the roadsides (a long-term procedure) is not possible because those trees are protected.

14.7.1.6 Segment: "Via Poli"

Tab. 14.27: Characteristics of the segment: "Via Poli" (model variables such as AADT, segment length, specific geometric element and observed crashes).

Model Variables	*Homogenous part 1*
AADT	920
Segment length [km]	0.048
Horizontal alignment element	Tangent
Number of crashes	0

Fig. 14.66: View of the segment: "Via Poli" (photo source Google Earth).

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, the segment Via Poli shows the following problems:

- there are slight pavement cracks at the middle of the segment.

The short-term countermeasures are collected in only one set:

- set B– Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - installation of the stop traffic sign with integrative distance panel.

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

Fig. 14.67: Short-term countermeasures Via Poli.

14.7.1.7 Segment: "Via Niceforo"

Tab. 14.28: Characteristics of the segment: "Via Niceforo" (model variables such as AADT, segment length, specific geometric element and observed crashes).

Model Variables	*Homogenous part 1*	*Homogenous part 2*	*Homogenous part 3*
AADT	1260	1260	1260
Segment length [km]	0.022	0.042	0.08
Horizontal alignment element	Tangent	Tangent	Tangent
Number of crashes	1	1	0

Fig. 14.68: View of the segment: "Via Niceforo" (photo source Google Earth).

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, this homogenous part of Via Niceforo shows the following problems:

- 2 crashes occurred: 1 in the first homogenous part 22 m long, 1 in the second homogenous part 42 m long;
- the two crashes were both angle collisions due to high speed or distracted driving, in which the travelling vehicle impacted another vehicle getting out of the parking;
- there are slight cracks of the road pavement surface in the first part of the segment, while the terminal part is affected by brushwood and emerging tree roots;
- vehicles park on-street on both sides of the road even if there are no signed spots, thus implying that shoulders are practically absent;
- trash cans on the road edges are often moved out to carve out new parking spots;
- the lane width is slightly lower than the minimum allowed by the Italian standards D.M. 6792/2001[3] for "F" category roads (2.65 m instead of 2.75 m).

Two sets have then been proposed for the short-term interventions:

- set A - Short-term – Speed:
  - transverse optical speed bars, to reduce the vehicle speeds on this road (residential area);
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - installation of give-way and stop vertical signs with integrative distance panels;
  - removal of brushwood and emerging tree roots;
  - relocation of trash cans in areas delimited by appropriate markings.

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

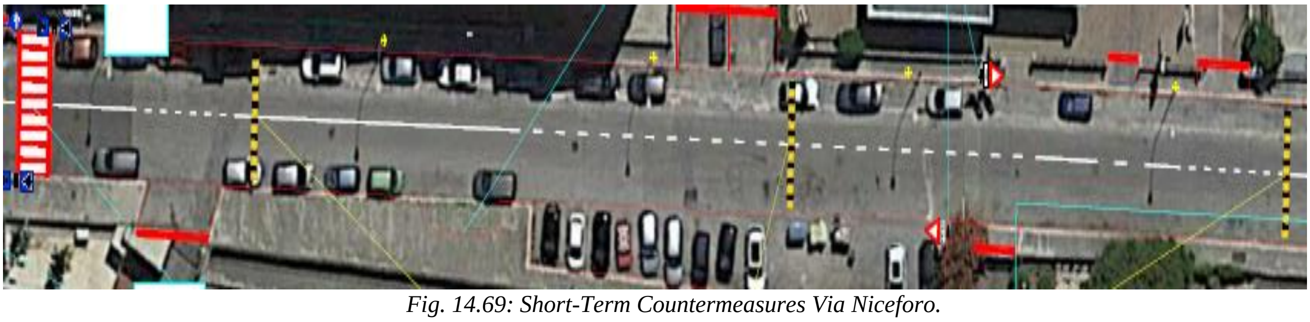

Fig. 14.69: Short-Term Countermeasures Via Niceforo.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- reduction of the sidewalk width in the first part of the segment in order to increase the sight distance related to the driveway;
- enlargement of the sidewalk width in the second part of the segment to modify the arrangement of parking spots, from angle to parallel parking.

Fig. 14.70: Long-Term Countermeasures Via Niceforo.

14.7.1.8 Intersection I

Tab. 14.29 Characteristics of the Intersection I (model variables such as the AADT on the main segment and on the secondary segment, number of crashes, type of intersection).

Model variables	
AADT- main road	20365
AADT- secondary road	15200
Number of crashes	18
Intersection type	4-legged signalized intersection

Fig. 14.71: View of the Intersection I (photo source Google Earth).

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From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, the intersection I shows the following problems:

- there were 18 crashes;
- the crashes were: 6 angle collision, 3 pedestrian hits; 5 sideswipe collisions, 4 rear-end crashes. even if the traffic lights work only from 7:00 a.m. to 11:00 p.m., only 2 crashes out of 18 occurred with the traffic lights off. the rear-end crashes are essentially caused by a poor perception of the road environment and safe headways by the users, who have also probably incorrectly estimated the "yellow light" duration. The sideswipe collisions are mainly caused by the absence of vertical traffic signs warning about the different lane directions, which could be wrongly occupied by vehicles. the angle collisions can be caused by the disregard of traffic lights (i.e., running the red light), instead pedestrian collisions are mainly due to pedestrians illegally crossing the road;
- in some parts, shoulders are absent or their width is insufficient;
- in the northern intersection quadrant, the on-street give-way markings are absent;
- the vertical give-way traffic sign, in the northern quadrant, is partially covered/hidden by tree branches close to the traffic light;
- the road surface shows cracks and ruts in small areas of the northern and southern quadrants;
- in the western quadrant, there is an obstacle in the median.

Two sets have then been proposed for the short- term interventions:

- set A - Short-term – Speed:
    - installation of speed cameras, to enhance enforcement (infra-red cameras for red light violations).
- set B – Short-term – Extraordinary maintenance:
    - remaking of the road pavement surface and consequently of the horizontal road markings;
    - installation of barriers on sidewalks to prevent illegal crossings by pedestrians;
    - implementation of high visibility and two-steps pedestrian crossings, with pedestrian refuge island;
    - installation of pedestrian traffic lights provided with time countdown;
    - installation of an adaptive traffic light to reduce traffic congestion;
    - removal of the obstacle on the median in the western quadrant;
    - pruning of vegetation close to the traffic lights;
    - improving visibility of vertical traffic signs;
    - forbid vehicles stopping at the intersection, thanks to zebra patterns and rubber curbs 15 cm tall.

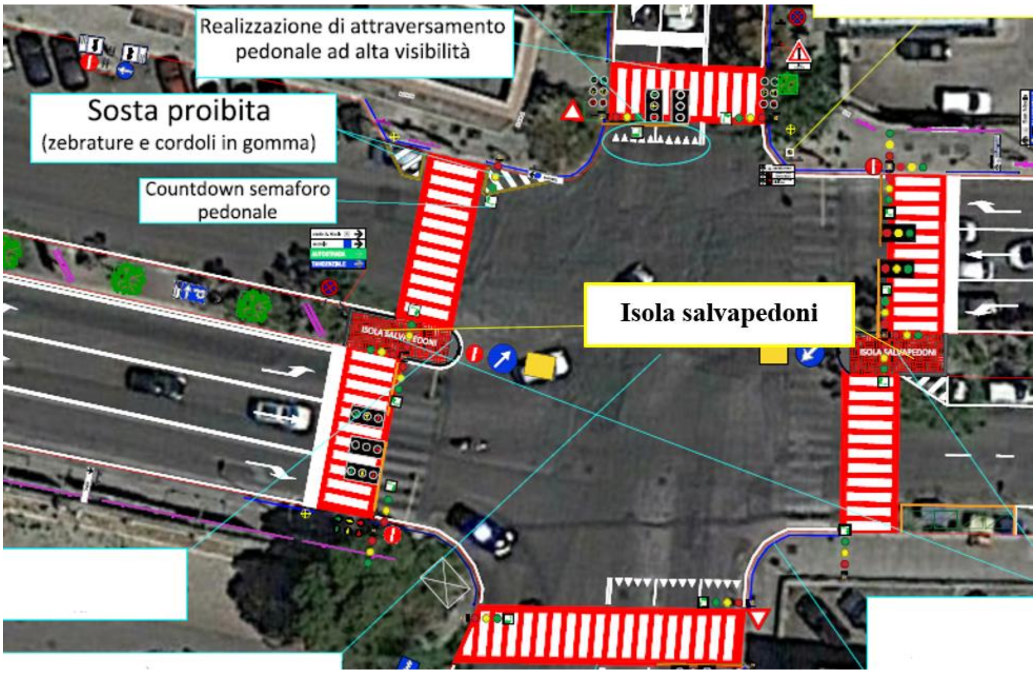

Fig. 14.72: Short-Term countermeasures - Intersection I (Sosta proibita = prohibited parking/realizzazione di rampe per pista ciclabile = construction of cycle path ramps/isola salvapedoni= pedestrian refuge island/Realizzazione di attraversamento pedonale ad alta visibilità = Implementation of high visibility and two-steps pedestrian crossings/Countdown semaforo pedonale = Countdown pedestrian traffic light/zebrature e cordoli in gomma = Zebra patterns and rubber kerbs).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- implementation of advanced stop lines (bike boxes) to accurately separate cyclists and vehicles in the stop area at the traffic lights;
- new cycle/pedestrian crossings provided with adequate vertical traffic signs.

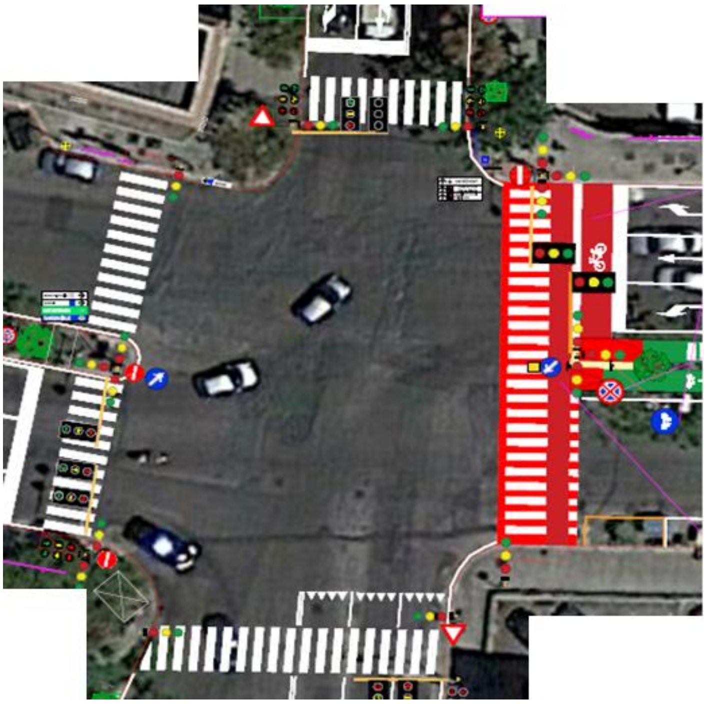

Fig. 14.73: Long-Term countermeasures Intersection I.

14.7.1.9 Intersection II

Tab. 14.30: Characteristics of the Intersection II (model variables such as the AADT on the main segment and on the secondary segment, number of crashes, type of intersection).

Model variables	
AADT- main road	7500
AADT- secondary road	920
Number of crashes	1
Intersection type	3-legged unsignalized stop-controlled intersection

Fig. 14.74: View of the Intersection II (photo source Google Earth).

According to the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, the

Intersection II shows the following problems:

- there was 1 crash;
- the crash occurred is an angle collision type caused by the high speed of the vehicle coming from Via Poli which did not respect the stop signal;
- in the northern quadrant, the on-street stop markings are faded due to the ruts and the alligator cracking in the road surface;
- in the northern quadrant, the crosswalks are faded due to the ruts in the road surface;
- in the eastern and western quadrants, there are no vertical traffic signs to warn about the crosswalk presence.

Two sets of short-term interventions have been proposed:

- set A - Short-term – Speed:
  - installation of transverse optical speed bars while approaching to the intersection in order to reduce the vehicle speeds on the road segment approaching to the intersection.
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the horizontal road markings;
  - improvement of the visibility of vertical traffic signs;
  - installation of barriers on sidewalks to prevent illegal crossings by pedestrians;
  - implementation of high visibility and two-step pedestrian crossings, provided with refuge islands, in the eastern and western quadrants;
  - installation of LED vertical traffic signs at the crosswalks in the Eastern and Western quadrants;
  - installation of high visibility crosswalks in the Northern quadrant.

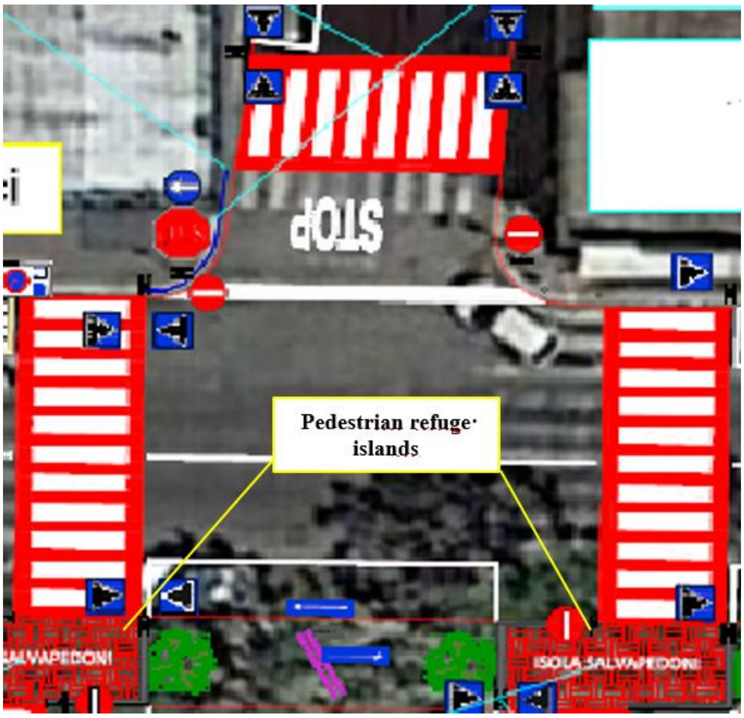

Fig. 14.75: Short-Term countermeasures Intersection II (pedestrian refuge islands).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

  - New cycle/pedestrian crossings provided with adequate vertical traffic signs.

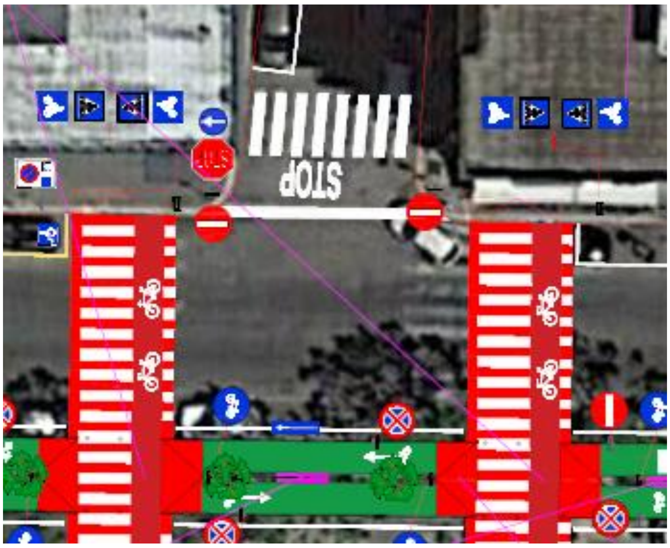
Fig. 14.76: Long-Term countermeasures Intersection II.

14.7.1.10 Intersection III

Tab. 14.31: Characteristics of the Intersection III (model variables such as the AADT on the main segment and on the secondary segment, number of crashes, type of intersection).

Model variables	
AADT-main road	6500
AADT-secondary road	1260
Number of crashes	1
Intersection type	3-legged unsignalized give-way controlled intersection


Fig. 14.77: View of the Intersection III (photo source Google Earth).

Results from the analysis of the collision diagrams, the in-site inspection outputs and the condition diagrams, show the following critical issues in the Intersection III:

- there was 1 crash;
- the crash was a pedestrian collision caused by illegal crossings by pedestrians at the intersection;
- in the southern quadrant, there are no crosswalks;
- in the southern quadrant, the on-street give-way markings are faded due to road surface damages at the intersection;
- in the eastern and western quadrants, there are no vertical traffic signs to warn about the crosswalk presence.

Hence, two sets of short-term interventions have been proposed:

- set A - Short-term – Speed:

- installation of transverse optical speed bars while approaching to the intersection in order to reduce the vehicle speeds on the road segment approaching to the intersection;
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;
  - Improvement of the visibility of vertical traffic signs;
  - Installation of barriers on sidewalks to prevent illegal crossings by pedestrians;
  - Implementation of high visibility and two-step pedestrian crossings, provided with pedestrian refuge island, in the eastern and western quadrants;
  - Installation of LED vertical traffic signs at the crosswalks in the eastern and western quadrants;
  - Installation of high visibility crosswalks in the southern quadrant.

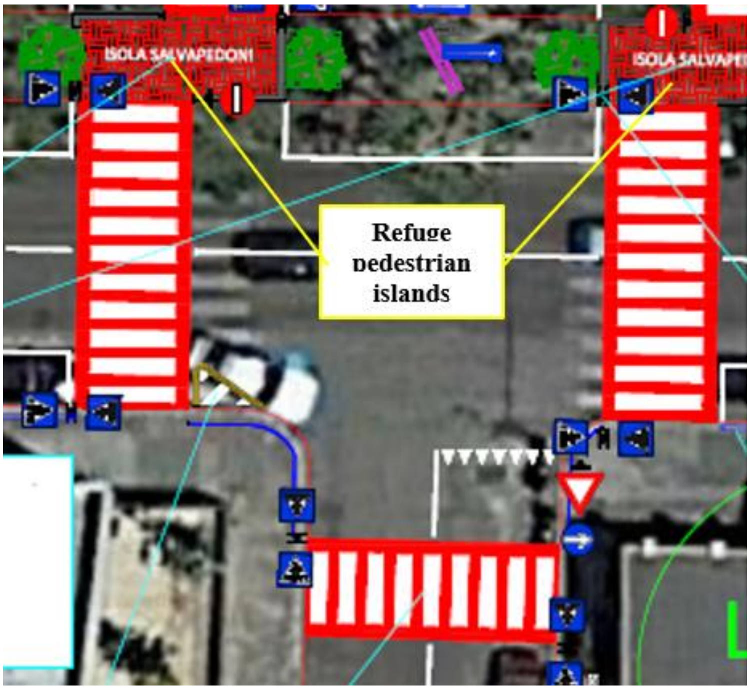

Fig. 14.78: Short-Term countermeasures Intersection III (isola salvapedoni= pedestrian refuge island).

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:
  - New cycle/pedestrian crossings provided with adequate vertical traffic signs.


Fig. 14.79: Long-Term countermeasures Intersection III.

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14.7.1.11 Intersection IV

Tab. 14.32: Characteristics of the Intersection IV (model variables such as the AADT on the main segment and on the secondary segment, number of crashes, type of intersection).

Model variables	
AADT- main road	15045
AADT- secondary road	5265
Number of crashes	8
Intersection type	unsignalized give-way controlled intersection

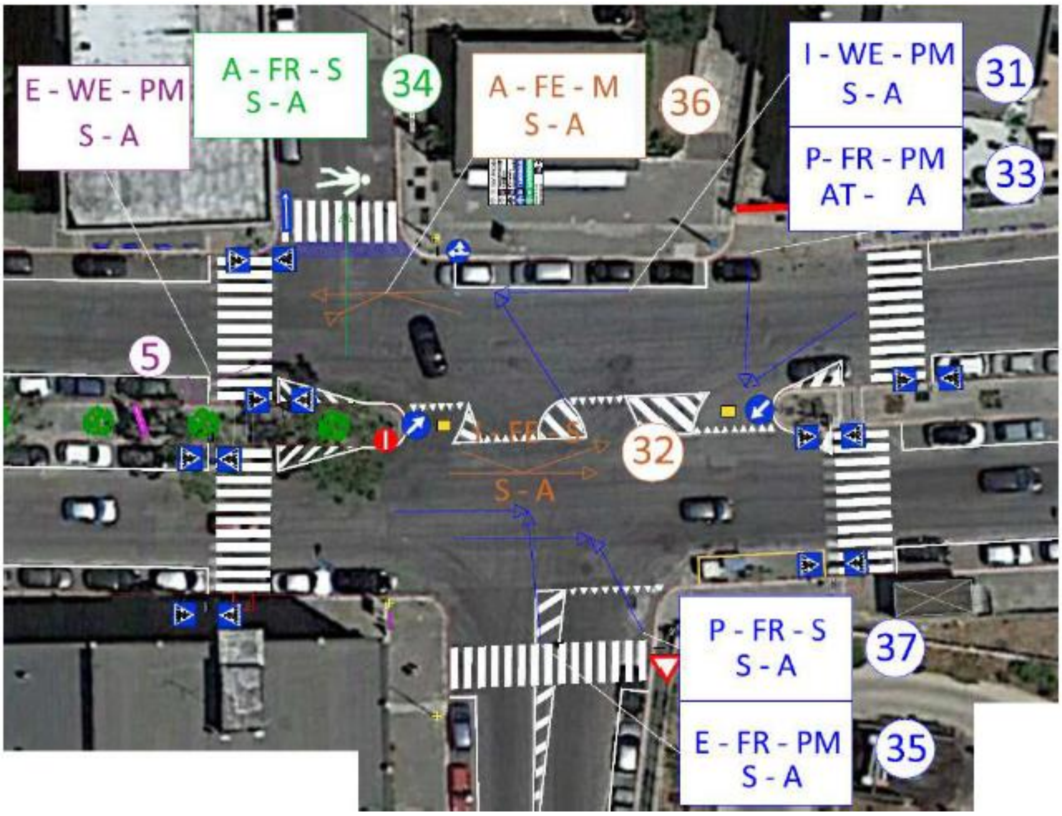

Fig. 14.80: View of the Intersection IV (photo source Google Earth).

From the analysis of the collision diagrams, in-site inspection outputs and the condition diagrams, the Intersection IV shows the following problems:

- there were 7 crashes;
- the 7 crashes were: 4 angle collisions possibly due to the disregard of the vertical give-way sign in Via Lioce and of the road markings; irregular crossings by pedestrians at the intersection; 1 pedestrian collision in the Northern quadrant while the pedestrian was regularly crossing the street; 2 sideswipe collisions in presence of high speeds;
- partial absence of shoulders;
- there are on-street painted traffic islands which are not adequately respected by the users approaching to the intersection;
- in the northern and southern quadrants, there are no vertical traffic signs to warn about the crosswalk presence.
- the road surface shows degradations and damages in the northern quadrant;
- parking is not regulated close to the intersection.

Hence, two sets of short-term interventions have been proposed:

- set A - Short-term – Speed:
  - Installation of transverse optical speed bars while approaching the intersection in order to reduce the vehicle speeds on the road segment approaching to the intersection;
- set B – Short-term – Extraordinary maintenance:
  - remaking of the road pavement surface and consequently of the road markings;

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- improvement of the visibility of vertical traffic signs;
- installation of barriers on sidewalks to prevent illegal crossings by pedestrians;
- implementation of high visibility and two-steps pedestrian crossings, provided with pedestrian refuge island and LED vertical traffic signs in the eastern and western quadrants;
- installation of high visibility crosswalks in the northern and southern quadrants;
- installation of vertical traffic signs to warn about crosswalks in the southern quadrant;
- installation of rubber curbs 15 cm tall to delimit the on-street painted traffic islands;
- implementation of a vertical stop traffic sign instead of the current give-way traffic sign in Via Lioce.

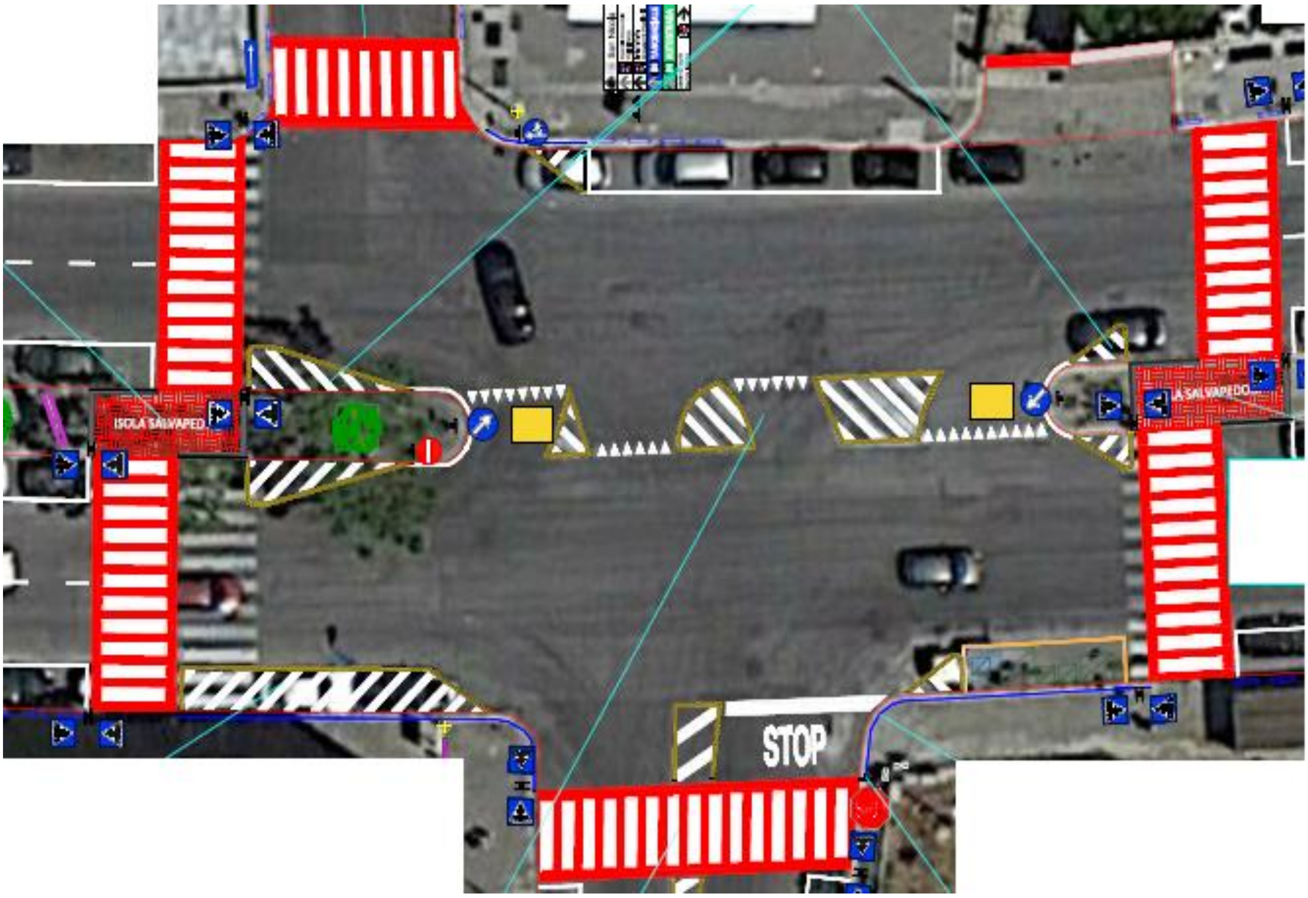

Fig. 14.81: Short-Term countermeasures - Intersection IV.

The ordinary maintenance of the road is compulsory in order to avoid potholes, cracks and ruts. It is necessary as well for the horizontal and vertical traffic signs in order to ensure their function.

The Long-term countermeasures are summarized in the following set, Set C – Long-term:

- installation of a compact roundabout with an inscribed circle diameter of 28 m; with 2 entry lanes (total width: 6.00 m) and 1 exit lane (4.50 m wide) in the eastern-western quadrants; with 1 entry lane (3.50 m wide) and 1 exit lane (4.50 m wide) in the southern quadrant and with 1 exit lane (4.50 m wide) in the northern quadrant. The total roundabout carriageway width is 8.50 m. Adequate transverse optical speed bars in approach to the intersection are drawn. Moreover, the roundabout must be provided with appropriate vertical traffic signs and road markings (Italian road regulations);
- installation of a one-way cycle path separated from the roadway and provided with appropriate vertical traffic signs and markings;
- implementation of cycle/pedestrian crossings 5.00 metres before the edge of the roundabout. These crossings are provided with a row of give-way squares (L = 0.50 m), crosswalks (L = 2.50 m) and one cycle lane (L = 1.50 m). They must be provided with LED vertical traffic signs;
- installation of bollards and barriers on sidewalks to protect pedestrians and to avoid their illegal crossings.

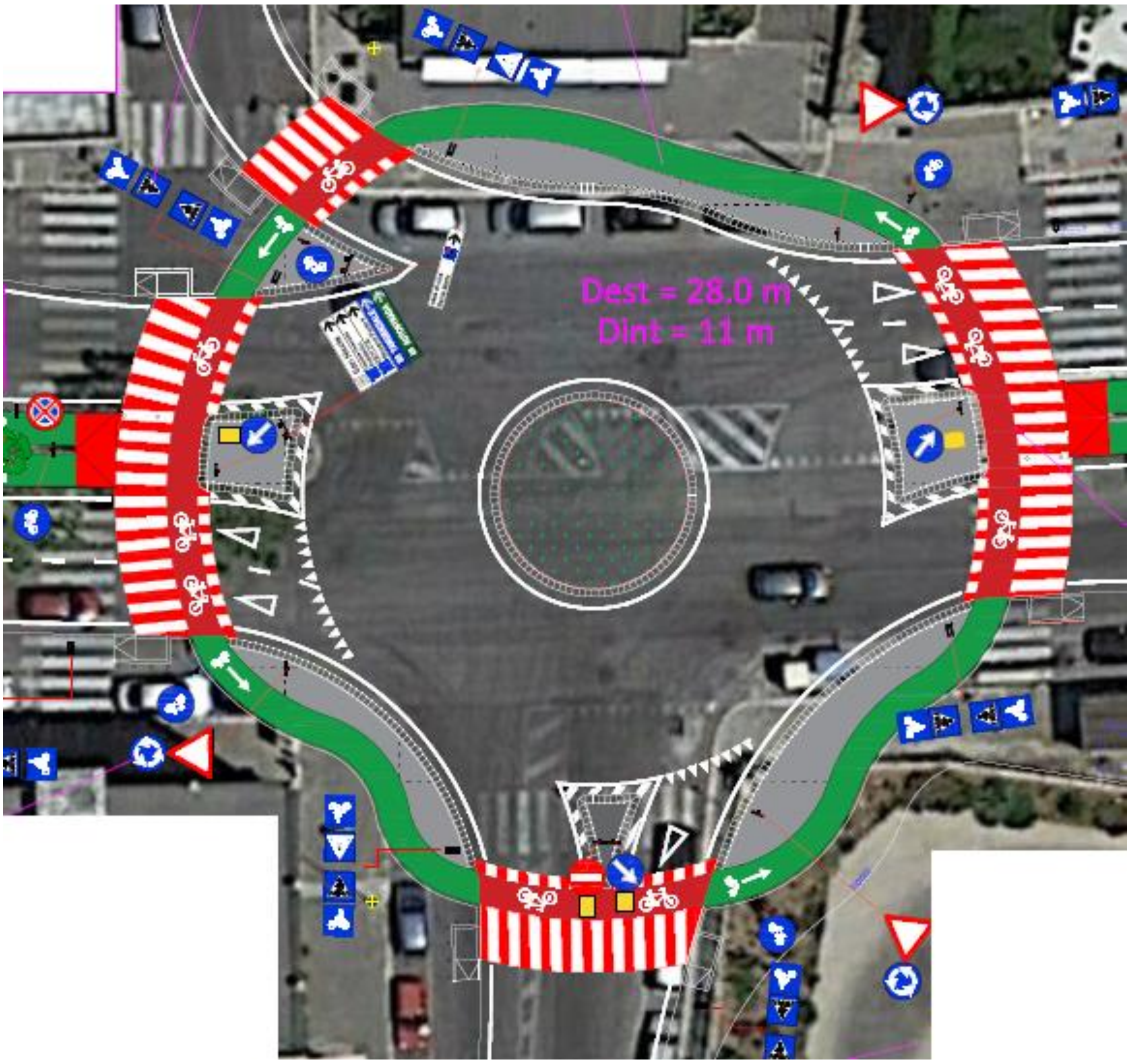

Fig. 14.82: Long-Term countermeasures - Intersection IV.

14.7.2 CMFs for possible sets of countermeasures

After the selection of the countermeasures as based on the analysis of crash data, collision diagrams, condition diagrams, related in-site inspections, crash locations and reconstructed circumstances, their future effects on crashes at each site was analysed.

Different combinations of countermeasures were used with the aim of reducing the crash frequency:

1. combination 1 – Set A (Short-term – Speed);
2. combination 2 – Set B (Short-term – Extraordinary maintenance);
3. combination 3 – Set A + Set B;
4. combination 4 – Set C (Long-term);
5. combination 5 – Set B + Set C.

The combinations 3 and 5 are not further considered because the single countermeasures included in the different sets may collide one with each other (for example, converting the intersection into a roundabout is not compatible with implementing rubber curbs on the on-street painted traffic islands).

Since the effect of the introduction of cycle paths is hard to be assessed, two different scenarios were studied: one in which the cycle path implementation will bring slight global benefits to road safety (CMF equal to 0.9) and another scenario in which the cycle path implementation will have no effects on crashes (CMF equal to 1.0). Hence, besides the long-term combinations 4 and 5 (cycle path CMF = 0.9), the combinations 6 and 7 were added (cycle path CMF =1.00).

However, it is still unclear the benefit of cycle path on the road safety performances[2]. Nevertheless, even if several analyses have been carried out in order to assess the safety performances of cycle paths, mixed results were found about the safety effects of bike lanes, roundabouts and separated bike tracks for what concerns cyclists' crashes and injuries.

The inconsistency of these results may depend on geometric variables of the road, the penetration rate of cyclists, the culture towards the vulnerable road users and so forth. For example, an increase of vulnerable road user mobility has led in several cases to a reduction of crashes[12]. In fact, as argued in Colonna et al. (2018)[13], in Bari (Italy), there are only 20 km long cycle paths. Whereas, in a city like Malmo (Sweden), there are hundreds of kilometres of cycle paths. However, there are evident differences between Italy and Sweden in macro-level safety statistics (cyclist death rate is 4 per million inhabitants in Italy, the double of the Swedish rate) [14].

Implementing a cycle path as a long-term countermeasure sometimes requires other complementary long-term changes (roundabouts, new road geometry), creating more complex situations to be evaluated by a CMF value in terms of safety performances. So, there is not yet a single, valid and quantifiable CMF associated to cycle paths.

If the consequences on crashes of a countermeasure are not clearly quantifiable, their safety performances too cannot be quantified. In case of cycle path implementation three different approaches have been tested, though.

The first approach could involve implicitly assessing the benefits of the sustainable mobility on crash occurrence, so the cycle path is excluded from the economic assessment of the implementations.

The second approach, more conservative, consists of considering the implementation costs but not the safety benefits (CMF = 1.0).

The third one, the most complete approach and the one used in this example consists of the parallel analysis of two scenarios: one with no benefits introduced by the cycle path (CMF = 1.0) and the other that considers a low benefit of the cycle path implementation (CMF = 0.9).

Then, these two scenarios are compared to understand the differences from the baseline case (CMF =1.0). The expected crash frequency (by means of the EB-method) was calculated for the current conditions, in the early stages of the analysis ($N_{Expected/Before}$). After, the expected crash frequency was calculated for the design conditions after having assumed the implementation of the countermeasures ($N_{Expected/After}$).

The difference between the two values of the expected crash frequency is a measure of the benefits provided by the countermeasures to road safety:

$$\Delta N_{Expected} = N_{Expected/Before} - N_{Expected/After} \qquad \text{(Eq. 14-13)}$$

A spreadsheet was used for the calculation of the difference in the expected crash frequency for each segment and intersection studied. An example of output from the calculation is reported as follows.

Tab. 14.33: Example of calculation of the difference in the expected crash frequency ($\Delta N_{expected\_\#\#}$)between the expected crash frequency in the current conditions (before countermeasure implementation, $N_{expected\_\#\#\_current\ conditions}$) and after the countermeasure implementation ($N_{expected\_\#\#\_Combination2}$ in this example), for each type of crash (KABC: fatal + injury crashes; O: Property damage only crashes; All: KABCO), segment and intersection – Combination 2 example.

	Road Segment							*4-Legged Intersection*		*3-Legged Intersection*	
	T1N. G. XXIII	*T1S. G. XXIII*	*T2N. G. XXIII*	*T2S. G. XXIII*	*Flacco*	*Poli*	*Niceforo*	*Flacco/ G. XXIII*	*Lioce/ G. XXIII*	*Poli/ G. XXIII*	*Niceforo/ G. XXIII*
	1	*2*	*3*	*4*	*5*	*6*	*7*	*1*	*4*	*2*	*3*
$N_{expected}$_KABC_Current	0.933	0.827	1.360	1.952	2.530	0.131	0.651	4.180	9.095	1.821	0.745
$N_{expected}$_KABC_Comb.2	0.451	0.400	0.755	1.084	0.806	0.091	0.355	1.651	2.390	1.364	0.476
$\Delta N_{expected}$_KABC	0.482	0.427	0.605	0.868	1.724	0.040	0.297	2.529	6.706	0.457	0.269
$N_{expected}$ _O_ Current	6.844	6.066	9.973	14.316	18.552	0.959	4.777	30.656	66.700	13.355	5.461
$N_{expected}$_O_Comb.2	3.309	2.933	5.537	7.947	5.911	0.669	2.602	12.111	17.524	10.003	3.488
$\Delta N_{expected}$_O	3.535	3.133	4.437	6.369	12.642	0.290	2.175	18.545	49.176	3.352	1.973
$N_{expected}$_All_ Current	7.777	6.894	11.333	16.268	21.082	1.090	5.428	34.836	75.795	15.176	6.206
$N_{expected}$_All_Comb. 2	3.760	3.333	6.292	9.031	6.717	0.760	2.957	13.762	19.913	11.367	3.964
$\Delta N_{expected}$_All	4.017	3.561	5.042	7.237	14.366	0.330	2.472	21.074	55.882	3.809	2.242

12 Jacobsen P. L. (2003), "Safety in numbers: More walkers and bicyclists, safer walking and bicycling", *Injury Prevention*, 9, 205-209.

13 Colonna P., Intini P., Berloco N., Fedele V., Masi G., Ranieri V. (2019), "An Integrated Design Framework for Safety Interventions on Existing Urban Roads. Development and Case Study Application", *Safety*, 5(1), 13.

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14.8 Benefit-cost analysis

The differences between the expected crash frequency before and after the countermeasures can be differentiated into changes in the expected crash frequency for Fatal+Injury crashes (FI) and for Property Damage Only crashes (PDO). This difference must be converted into a monetary value for each year of the countermeasure lifetime, to assess the benefits. In the urban context, their average lifetime can be assumed as equal to 10 years for both short-term and long-term countermeasures. The monetary conversion can be made by considering the social costs provided in the report by the Ministry of Infrastructures and Transport (2011)[15].

Tab. 14.34: Example of calculation of the present values of benefits for the Combination 2 of countermeasures. Crash cost for each category (KABC and O) times the difference between the expected crashes before and after the countermeasure implementation ($\Delta N_{expected\_\#\#\#\_Combination2}$ in this example) is equal to the saved amount of money thanks to the countermeasure, for each crash type ($AM_{\_O\_Combination2}$ and $AM_{\_KABC\_Combination2}$).

Combination 2 - Short-Term (Set 2)						
Counter-measure lifetime [Years]	*ΣΔNexpected_KABC_Comb.2 (1 year)*	*Crash cost_KABC*	*AM_KABC_Comb.2 (1 year)*	*ΣΔNexpected_O_Comb.2 (1 year)*	*Crash cost_O*	*AM_O_Comb.2 (1 year)*
10	14,40	309863 €	4 463128,93 €	105,63	10 986,00 €	1 160 408,05 €

Calculation of The Present Benefit over the Countermeasure Lifetime							
Countermeasure lifetime [years (actual-progressive)]		PA, i,y (discount rate = 3.5 %)	AM_KABC	AM_O	Present value_ KABC_Comb.2	Present value_O_Comb.2	
2019	1	1.00	4 463 128,93 €	1 160 408,05 €	4 463 128,93 €	1 160 408,05 €	
2020	2	1.90	4 463 128,93 €	1 160 408,05 €	8 478 580,48 €	2 204 420,53 €	
2021	3	2.80	4 463 128,93 €	1 160 408,05 €	12 504 067,06 €	3 251 042,10 €	
2022	4	3.67	4 463 128,93 €	1 160 408,05 €	16 393 426,08 €	4 262 270,68 €	
2023	5	4.52	4 463 128,93 €	1 160 408,05 €	20 151 260,88 €	5 239 303,12 €	
2024	6	5.33	4 463 128,93 €	1 160 408,05 €	23 782 019,13 €	6 183 295,81 €	
2025	7	6.11	4 463 128,93 €	1 160 408,05 €	27 289 998,13 €	7 095 366,05 €	
2026	8	6.87	4 463 128,93 €	1 160 408,05 €	30 679 349,82 €	7 976 593,33 €	
2027	9	7.61	4 463 128,93 €	1 160 408,05 €	33 954 085,74 €	8 828 020,65 €	
2028	10	8.32	4 463 128,93 €	1 160 408,05 €	37 118 081,81 €	9 650 655,75 €	
				Total (KABC + O = ALL)	214813998,05 €	55851376,06 €	270665374,11 €

After, the obtained benefit must be converted into present values (the assumed discount rate is: 3.5%). The monetary benefit is achieved by considering the sum of the changes in the expected mean crash frequency on each road network element, for each year of countermeasure implementation, multiplied by the estimated cost of a single crash (for different severities). The estimated cost of a crash depends on its severity. In Italy, each FI (Fatal+Injury) crash has an average social cost of € 309863,00; while a PDO crash has an average social cost of € 10986,00. The present value of the benefit for each year of the countermeasure implementation is calculated considering the discount rate (3.5% in this case). The benefit present value is then calculated as follows:

$$PV_{Benefits} = Total\ Annual\ Monetary\ Value\ (PA, i, y) \quad \text{(Eq. 14-14)}$$

where (PA, i, y) is the conversion factor to the present value for a series of uniform annual amounts:

$$(PA, i, y) = \frac{(10+i)^y - 1.0}{i\ (10+i)^y} \quad \text{(Eq. 14-15)}$$

where:

- i is the discount rate, 3.5 %;
- y is the countermeasure lifetime (years).

[15] Ministero delle Infrastrutture e dei Trasporti, Dipartimento per i Trasporti, la Navigazione ed i Sistemi Informativi e Statistici, Direzione Generale per la Sicurezza Stradale (2010), *Studio di valutazione dei Costi Sociali dell'incidentalità stradale*, Report.

Another spreadsheet is used to calculate the benefit present values for each set of countermeasures. An example of the output obtained from the calculation is reported below.

After the present values of benefits from countermeasures are computed, the costs of implementation of each set of countermeasures are estimated as well.

The estimated costs should include the ordinary maintenances during the lifetime (10 years), trying to foresee the number of interventions over this time period. This calculation is crucial because the interventions made during the countermeasure lifetime should be converted into present values. The total cost of a combination is essentially the sum of the implementation costs of the countermeasures and of maintenance costs (present values).

An example of estimation of costs related to each set of countermeasures is reported below.

Tab. 14.35: Example of estimated costs (present values) for the Combination 2 of countermeasures. The total cost is given by the sum of the all Present Values costs (obtained by the product of the Maintenance COST_$_{Combination2}$ and the conversion factor to the present value, PA, i, y) due to the maintenance over the countermeasure lifetime and the cost for firstly implementing the countermeasures (COST_$_{Combination2}$).

Countermeasure lifetime [Years]	*Cost Combination 2*	*Maintenance Cost Combination 2*	*Maintenance Cost Combination 2 (1 year)*
10	€ 353714,900	€ 521720,700	52172,07 €

Calculation of The Present Benefit over the Countermeasure Lifetime

Countermeasure lifetime [years (actual-progressive)]		PA, i,y (discount rate = 3,5 %)	Maintenance Cost Combination 2 (1 year)	Present Value Cost Combination 2	
2019	1	1.00	52 172,07 €	52 172,07 €	
2020	2	1.90	52 172,07 €	99 110,98 €	
2021	3	2.80	52 172,07 €	146 167,20 €	
2022	4	3.67	52 172,07 €	191 632,15 €	
2023	5	4.52	52 172,07 €	235 559,63 €	
2024	6	5.33	52 172,07 €	278 001,64 €	
2025	7	6.11	52 172,07 €	319 008,42 €	
2026	8	6.87	52 172,07 €	358 628,49 €	
2027	9	7.61	52 172,07 €	396 908,75 €	
2028	10	8.32	52 172,07 €	433 894,52 €	
		Total Cost (Present total costs including the implementation costs)		*2511083,84 €*	*2864798,74 €*

14.8.1 Ranking of projects

The economic evaluation can be assessed through either the NPV (Net Present Value) or the BCR (Benefit-Cost ratio) method. The NPV is the difference between the obtained benefits from the implementation of each set of countermeasures and the costs to implement it; the BCR is the ratio between benefits and costs of the countermeasures. If the NPV is positive, the countermeasure selection is justified, as well as the BCR must be greater than 1. The calculation of the NPV and BCR values for each of the considered combinations of sets of countermeasures is shown below.

Tab. 14.36: NPV and BCR values calculated for each combinations of sets of countermeasures (cycle path CMF =0.9).

Scenario: Cycle Path Cmf = 0.9	*Present Costs*	*Present Benefits*	*NPV*	*BCR*
Combination 1 Set A	208 359,87 €	91 518 705,71 €	91 310 345,84 €	439.2
Combination 4 Set C	909 862,73 €	170 241 852,85 €	169 331 990,12 €	187.1
Combination 2 Set B	2 864 798,74 €	270 665 374,11 €	267 800 575,37 €	94.5
Combination 3 Set A+B	3 073 158,61 €	318 558 956,16 €	315 485 797,55 €	103.7
Combination 5 Set B+C	3 774 252,47 €	180 984 026,73 €	177 209 774,26 €	48.0

Tab. 14.37: NPV and BCR values calculated for each combinations of sets of countermeasures (cycle path CMF =1.0).

Scenario: Cycle Path Cmf = 1	*Present Costs*	*Present Benefits*	*NPV*	*BCR*
Combination 1 Set A	208 359,87 €	91 518 705,71 €	91 310 345,84 €	439.2
Combination 6 Set C	909 862,73 €	164 050 401,47 €	163 140 538,74 €	180.3
Combination 2 Set B	2 864 798,74 €	270 665 374,11 €	267 800 575,37 €	94.5
Combination 3 Set A+B	3 073 158,61 €	318 558 956,16 €	315 485 797,55 €	103.7
Combination 7 Set B+C	3 774 252,47 €	177 253 558,99 €	173 479 306,52 €	47.0

Some remarks:

- The NPV value calculated for the combination 3 (sum of the short-term sets of countermeasures) is the greatest in both cases considered (cycle path CMF = 0.9 or 1.0);
- The BCR value calculated for the combination 1 (short-term set A) is the greatest in both cases considered (cycle path CMF = 0.9 or 1.0).

The ranking list obtained according to the NPV or the simple BCR method explained may be less reliable than the incremental analysis derived from the comparison of the BCR values.

The incremental analysis (one of the methods suggested by the HSM, 2010[6]) compares couples of combinations, based on the difference between the benefit of the second cheapest project and the benefit of the cheapest project (as explained in the section 13.3.10.3).

The calculations for an example scenario (cycle path CMF = 0.9) is shown below.

Tab. 14.38: Incremental BCR analysis (cycle path CMF = 0.9).

Scenario: Cycle Path Cmf = 0.9			
		Incremental Analysis	
Couples Of Combinations	*Δ Benefits (Cx - Cy)*	*Δ Costs (Cx - Cy)*	*Incremental Ratio*
C1 - C4	€ 78,723,147.15	€ 701,502.86	112.221
	C4 "wins"		
C4 - C2	€ 100,423,521.26	€ 1,954,936.01	51.369
	C2 "wins"		
C2 - C3	€ 47,893,582.05	€ 208,359.87	229.860
	C3 "wins"		
C3 - C5	-€ 137,574,929.43	€ 701,093.86	-196.229
	C3 "wins"		
		Incremental Analysis	
Couples Of Combinations	*Δ Benefits (Cx - Cy)*	*Δ Costs (Cx - Cy)*	*Incremental Ratio*
C1 - C4	€ 78,723,147.15	€ 701,502.86	112.221
	C4 "wins"		
C4 - C2	€ 100,423,521.26	€ 1,954,936.01	51.369
	C2 "wins"		
C2 - C5	-€ 89,681,347.38	€ 909,453.72	-98.610
	C2 "wins"		
		Incremental Analysis	
Couples Of Combinations	*Δ Benefits (Cx - Cy)*	*Δ Costs (Cx - Cy)*	*Incremental Ratio*
C1 - C4	€ 78,723,147.15	€ 701,502.86	112.221
	C4 "wins"		
C4 - C5	€ 10,742,173.87	€ 2,864,389.73	3.750
	C5 "wins"		
		Incremental Analysis	
Couples Of Combinations	*Δ Benefits (Cx - Cy)*	*Δ Costs (Cx - Cy)*	*Incremental Ratio*
C1 - C4	€ 78,723,147.15	€ 701,502.86	112.221
	C4 "wins"		

The final ranking lists of projects (alternative combinations of sets of countermeasures) are the following.

Tab. 14.39: Final ranking list of alternative projects (cycle path CMF = 0.9).

Scenario: Cycle Path CMF = 0.9	*Present Costs*	*Present Benefits*	*NPV*	*BCR*
Combination 4 Set C	909 862,73 €	170 241 852,85 €	169 331 990,12 €	187,1
Combination 5 Set B+C	3 774 252,47 €	180 984 026,73 €	177 209 774,26 €	48,0
Combination 2 Set B	2 864 798,74 €	270 665 374,11 €	267 800 575,37 €	94,5
Combination 3 Set A+B	3 073 158,61 €	318 558 956,16 €	315 485 797,55 €	103,7
Combination 1 Set A	208 359,87 €	91 518 705,71 €	91 310 345,84 €	439,2

Tab. 14.40: Final ranking list of alternative projects (cycle path CMF = 1.0).

Scenario: Cycle Path CMF = 1.0	*Present Costs*	*Present Benefits*	*NPV*	*BCR*
Combination 6 Set C	909 862,73 €	164 050 401,47 €	163 140 538,74 €	180,3
Combination 7 Set B+C	3 774 252,47 €	177 253 558,99 €	173 479 306,52 €	47,0
Combination 2 Set B	2 864 798,74 €	270 665 374,11 €	267 800 575,37 €	94,5
Combination 3 Set A+B	3 073 158,61 €	318 558 956,16 €	315 485 797,55 €	103,7
Combination 1 Set A	208 359,87 €	91 518 705,71 €	91 310 345,84 €	439,2

According to the incremental BCR method, the most convenient combination is the combination 4 (or 6, for CMF cycle path = 1), so the one which considers the Set C -Long-Term countermeasures.

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15.Future outlook and applications

The previous chapters deal with theoretical and practical aspects related to road safety engineering through a technical and scientific approach which paves the way for future outlooks and applications, explained in this chapter.

The main reference for how to quantify and assess safety improvements on road segments and intersections is the HSM. However, it was shown how the HSM method can be integrated with other local standards/guidelines. In fact, the protocol for safety interventions on existing roads suggested in this book was integrated with the Italian and European standards and regulations as well as with the recent research in this field.

Two examples of the adoption of this protocol were provided, one for the rural context and another for the urban context, relying on the HSM requirements. In this way, a wide and complete outlook of the possible interventions aimed at improving safety performances is provided. These examples may seem schematic, but they were preparatory for the aim of explaining the whole procedure for both segments and intersections in urban/rural contexts.

The HSM method requires the use of Safety Performance Functions (SPFs) and a preliminary stage of the application of this method is inevitably the calibration of the SPF for local conditions (or a local SPF development). An example of this procedure was provided in Chapter 12, were the Italian (in detail) and Scottish cases were described. Some examples of the estimation of local SPFs, highly advised by the HSM, were shown as well in the same chapter, for Italian rural and urban roads and for Scottish rural roads. Future developments of this approach will deal with the calibration and estimation of SPFs in other European countries, like Spain or Norway (where preliminary researches were attempted by the research team). It is important to note that the HSM method and the integrated protocol presented in this book can be potentially applied in every country, even if their application was shown in some specific countries, as examples.

Some research issues related to the infrastructure design with specific regard to safety were described in the previous chapters. Particularly the friction was dealt with in a detailed way, suggesting a new possible integrative check based on the Friction Diagram Method (FDM, Chapter 9). Future studies about the friction diagram can deal with the method calibration through the use of dynamic software on-board of vehicles, owned by specialized companies. The FDM was introduced in the proposed protocol for road safety interventions. However, given the need for further calibrating and validating the model, the proposed protocol can be used without the application of the FDM, which was shown for exploratory purposes.

In this book, a great attention was given to the human behaviour while driving, this is because of the universally recognized influence of the human factors on the crash occurrence. Topics like the Safety budget, differences between expectation and actual conditions, mental workload, attention are relevant in the driving process. The idea of drivers' familiarity to a determined route incorporate all the previous aspects. The familiarity has a great importance in road safety analyses, so it was highlighted in Chapter 4. The safety-related familiarity aspects should be deepened in regard to the improvement of road design procedures, which may consider this particular issue. Another research may lead to the development of a solid base to demonstrate the idea, developed in Chapter 4, that the external risk demands engineering interventions despite of what Wilde assessed. This is because the external risk is related to quick reactions which impede the internalization of the risk, so the road design must help the driver in this fast response through engineering solutions.

Moreover, it should be noted that the road safety could be completely revolutionised by the introduction of connected and automated vehicles in the market, with particular regard to the decrease in road crashes due to human factors and the possible changes in road geometric features, currently designed according to human factors. These two main issues should be considered in future safety research.

All governments of countries provided with a significant road network are orienting their investments towards the enhancement of existing infrastructures, particularly considering road safety.
This book is aimed to provide both the scientific background and an operational framework for safety enhancement of existing roads, applicable regardless of the specific country.

The scientific background presented includes the main theories about the crash phenomena and driver behavioural models; the basic concepts related to crashes and risk, the road safety management process and how to measure road safety performances and to identify high-risk sites, especially considering the *Highway Safety Manual* and the *European Guidelines*. A research focus on the crucial topic of tire-pavement road friction is provided. This book was strongly aimed to be an open access edition, in order to disseminate as much as possible the cutting-edge methods in road safety engineering. In fact, it is aimed at prioritising the human value thanks to the benefits from reduced severe crashes, possibly provided by the guidance of this book. Hence, it has a clear academic and educational purpose, being not intended for commercial purposes.

Based on the presented background, a new design protocol for safety interventions on existing roads is proposed. Moreover, two complete examples of design applications, both in the rural case (two-way two-lane rural road) and in the urban case (a small road network composed of segments and intersections) are provided. The main problems and possible solutions are addressed, considering also the issue of transferability of the methods to different contexts.

Why, as researchers and practitioners, do we work on road safety?
I would like to dedicate this work to all the "YOU" who were known by each of us and who lost their lives in a traffic crash, each of us could dedicate this book to a known person.
Each of us should undertake the task, through our apparently not important job, of giving back the opportunity of the gift of life to people, of being themselves and of being happy, to all the "YOU" that, even unconsciously, will avoid a traffic crash thanks to the methods that this book illustrates.
Someone will thank us.

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