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A Methodology for Sight Distance Analysis on Highways, Alignment Coordination, and their Relation to Traffic Safety

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César de Santos Berbel

Ingeniero de Caminos, Canales y Puertos

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PUERTOS

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Author:

César de Santos Berbel

*Ingeniero de Caminos, Canales y Puertos
Máster en Sistemas de Ingeniería Civil*

Supervisor:

María Castro Malpica

Dra. Ingeniera de Caminos, Canales y Puertos

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LOS VOCALES

EL SECRETARIO

A mi familia, por creer en mi.

A Lara, por mostrarme el camino.

Chaim Soutine: The Errant Road, 1939

Cole Swensen

as if a road could be otherwise but geometry
defies the man who is lost on the road that
the trees want to reach and reach down
to his walking on
along a verticality that defies
the requirements of normative perspective
and so he will reach, and the trees against chalk—
the gesture of the arm extended is central
to all Soutine's work be it a branch or an ache
or a split of the face, going off. In this case
can you say that a man is lost just because
you cannot distinguish him from the background.

"The only place success comes before work is in the dictionary."

Vince Lombardi

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Resumen

El diseño geométrico de las carreteras es un factor de máxima importancia para la seguridad de la circulación. En este sentido, la distancia de visibilidad es una herramienta fundamental tanto para establecer criterios de diseño geométrico de carreteras de nueva planta como para evaluar el trazado de carreteras ya existentes.

Las técnicas existentes de estimación de la distancia de visibilidad disponible producen resultados que pueden subestimar o sobreestimar la distancia de visibilidad disponible. En pocos casos se han aplicado sobre carreteras existentes métodos de alto rendimiento y probada precisión, y ninguno de ellos se ha aplicado a estudiar la relación con la seguridad viaria. La relación de la visibilidad disponible con la seguridad viaria no siempre es clara y la literatura muestra diversidad de resultados.

El presente trabajo desarrolla una metodología para el estudio de la distancia de visibilidad en tres dimensiones con el fin analizar el trazado de las carreteras desde el punto de vista de la seguridad viaria. A tal efecto, se aprovecha el alto rendimiento de un sistema de información geográfica, donde es posible llevar a cabo un análisis integral (visibilidad, consistencia, intensidad de tráfico, accidentes, etc.). En total se han examinado 11 tramos de carretera de calzada única y un tramo de autopista.

Primero se ha estudiado la relación entre distancia de visibilidad disponible y distancia de visibilidad de parada con distintos modelos de elevaciones, constatando que el empleo de modelos digitales de elevaciones que consideran, además del terreno, otros elementos como vegetación, dotaciones viarias o edificaciones, producen resultados más verosímiles. Las diferencias halladas entre los valores de distancia visibilidad disponible de los distintos modelos empleados fueron estadísticamente significativas.

A continuación se han empleado estos resultados conjuntamente con la teoría de fiabilidad para abordar el estudio de las implicaciones de la visibilidad disponible sobre la seguridad viaria. Se presentan dos métodos con la visibilidad de parada como función de estado límite para calcular la probabilidad de fallo, incorporando las variables implicadas mediante un enfoque probabilístico frente a los modelos deterministas clásicos todavía vigentes en el diseño geométrico de carreteras. El primer método plantea un estudio continuo de la visibilidad de parada a lo largo de un tramo de carretera. El segundo método analiza la visibilidad de parada en curvas en planta a partir de la visibilidad disponible obtenida en dos y tres dimensiones. Se ha observado que las diferencias en los resultados de probabilidad de fallo, derivadas de cada una de las técnicas de estimación de la visibilidad empleadas, las diferencias son aún más significativas que en los valores de visibilidad disponible. Los resultados de éste último análisis se utilizan además para ajustar un modelo lineal generalizado en curvas en planta que permita evaluar el riesgo de accidente por distancia de visibilidad insuficiente. Las variables de probabilidad y exposición empleadas resultaron estadísticamente significativas.

Finalmente se aborda la coordinación tridimensional del trazado en carreteras y sus posibles implicaciones en la seguridad viaria. En primer lugar se ha estudiado el efecto del uso de distintos modelos de elevaciones tanto en la detección de estos defectos del diseño como en los valores de los parámetros que los caracterizan en carreteras de calzada única. Además se han

identificado las variables de coordinación tridimensional del trazado que propician la existencia de reapariciones del trazado con mayor frecuencia y su relación con los parámetros característicos. Por otro lado, la coordinación del trazado en autopistas, donde los elementos de la planta, del alzado y la sección transversal tienen dimensiones mayores, se estudia para desarrollar criterios de diseño que permitan evitar la aparición de los principales defectos relacionados con la distancia de visibilidad y la perspectiva de la carretera.

Abstract

Geometric design is a factor of the utmost importance in highway safety. In this regard, sight distance is a fundamental principle to establish criteria both for the geometric design of new highways and to evaluate the alignment of in-service highways.

The outcome of most current sight distance estimation methods may underestimate or overestimate the available sight distance. Only in a few cases, high-performance methods of proven accuracy have been applied to in-service highways, and none of them has been applied to establish linkage between sight distance and highway safety. Moreover, the relationship between sight distance and highway safety is not always clear and the literature shows diversity of results.

The present work develops a methodology for the study of sight distance in three dimensions in order to assess the alignment of in-service highways through sight distance, from the point of view of highway safety. For that purpose, the high performance of a geographic information system is exploited, where more comprehensive analyses can be performed (including sight distance, design consistency, traffic volume, collision frequency, etc.). In total, 11 two-lane rural highways and a motorway section have been examined.

First, the difference between the available sight distance and the stopping sight distance has been studied considering different elevation models as inputs. It was observed that the use of digital elevation models that comprise other elements besides the bare ground, such as vegetation, road furniture or buildings, produce more lifelike results. The differences between the available sight distance values on the models used were found to be statistically significant.

These results were then used in conjunction with reliability theory to research the implications of available sight distance on highway safety. Two methods are presented contemplating the difference between the available sight distance and the stopping sight distance as the LSF to compute the probability of non-compliance. The variables involved are incorporated by means of a probabilistic approach instead of the classical deterministic models still in force in highway geometric design standards. The first method conducted a continuous study of the stopping sight distance along a highway section. The second one analyzes the stopping sight distance in horizontal curves from the available sight distance outputs in two and three dimensions. It was observed that the differences in the probability of non-compliance results, derived from each of the modelling methods contemplated, are statistically even more significant than in the values of the available sight distance. The results of the latter method are also used to fit a generalized linear model in horizontal curves that enables the evaluation of the accident risk caused by insufficient available sight distance. The probability and exposure variables used were statistically significant.

Finally, the three-dimensional alignment coordination of highways and its possible implications on traffic safety are broached. For two-lane highways, the effect of the use of different elevation models on both the detection of these design shortcomings and on their characteristic parameter values was studied. In addition, the alignment coordination variables which most often favor the existence of sight-hidden dips have been identified as well as their relation to the characteristic parameters. On the other hand, the three-dimensional alignment

coordination of motorways, where the horizontal and vertical alignments and the cross section are larger, is studied to develop design criteria that prevent the appearance of the main spatial alignment deficits related to sight distance and the perspective of the motorway.

Table of contents

Chapter 1. INTRODUCTION	1
1.1. Background	1
1.2. Motivation	2
1.3. Statement of the research problem	2
1.4. Objectives	3
1.5. Methodology	4
1.5.1. Elaboration of literature review.....	4
1.5.2. Methods, materials and procedure development.....	5
1.5.3. Pilot studies.....	5
1.5.4. Complete case study	6
1.5.5. Conclusions and future lines of research	6
Chapter 2. LITERATURE REVIEW.....	9
2.1. Introduction	9
2.2. Sight distance.....	9
2.2.1. General considerations	9
2.2.2. Required sight distance.....	10
2.2.3. Available sight distance.....	15
2.2.4. Inverse sight distance.....	22
2.3. Relation between sight distance and driving behavior	23
2.3.1. Driving behavior	23
2.3.2. Visual perception capabilities	24
2.4. Three-dimensional alignment coordination	25
2.4.1. General principles	25
2.4.2. Design flaws in the alignment	27
2.4.3. Evaluation of the 3D alignment coordination.....	35
2.5. Operating speed	43
2.5.1. Speed concepts	43
2.5.2. Speed prediction models	43
2.5.3. Creating operating speed profiles.....	46
2.6. Highway safety.....	47
2.6.1. Safety factors	48
2.6.2. Accident rates	50
2.6.3. Highway operation data.....	51
2.6.4. Statistical analysis of accident data	51
2.6.5. Research on sight distance and accidents on highways	56
2.6.6. Safety-related shortcomings in geometric design	59
2.6.7. Safety evaluation of highways	60
2.7. Reliability analysis.....	60
2.7.1. Reliability theory	61
2.7.2. Reliability applications in transportation.....	62
2.8. Alignment data extraction	63
2.9. Geographic information systems in highway research	65
2.10. Review summary.....	65

Chapter 3. METHODS AND MATERIALS.....	69
3.1. Computation of sight distance.....	69
3.1.1. Introduction	69
3.1.2. Road Sight Distance software	69
3.1.3. Creating a cartographic database	74
3.2. Alignment data extraction	84
3.2.1. Horizontal alignment data extraction	84
3.2.2. Vertical alignment data extraction	88
3.3. Selected highways	90
3.3.1. Introduction	90
3.3.2. Geometric characterization of the selected two-lane highways	90
3.3.3. Geometric characterization of the selected motorway section	94
Chapter 4. THREE-DIMENSIONAL SIGHT DISTANCE MODELLING	95
4.1. Introduction	95
4.2. Methodology	95
4.3. Results and discussion	96
4.3.1. Effect of roadside elements on sight distance results	96
4.3.2. Comparison of available sight distance results.....	98
4.4. Concluding remarks	104
Chapter 5. RELIABILITY ANALYSIS.....	107
5.1. Introduction	107
5.2. Pilot study using a sampling method.....	107
5.2.1. Methodology.....	107
5.2.2. Results and discussion	113
5.3. Study on curves using RT software	116
5.3.1. Determination of available sight distance input values.....	117
5.3.2. Operating speed model.....	121
5.3.3. Limit-state function.....	125
5.3.4. Random variables.....	125
5.3.5. Probability of non compliance computation	126
5.3.6. Results and discussion	127
5.3.7. Effect of grade.....	137
5.4. Safety performance functions incorporating reliability-based risk results	136
5.4.1. Statistical model.....	136
5.4.2. Results and discussion	138
5.5. Concluding remarks	139
Chapter 6. THREE-DIMENSIONAL ALIGNMENT COORDINATION	143
6.1. Introduction	143
6.2. Sight-hidden dips on two-lane highways.....	144
6.2.1. Methodology.....	144
6.2.2. Parameters of sight-hidden dips	145
6.2.3. Effect of sight distance modelling inputs on sight-hidden dip characterization	146
6.2.4. Effect of alignment parameters on sight-hidden dips	155
6.3. Development of standard spatial elements for motorways.....	162
6.3.1. Methodology.....	162

6.3.2. Results and discussion	164
6.4. Concluding remarks	168
Chapter 7. CONCLUSIONS AND FUTURE RESEARCH	171
7.1. Conclusions	171
7.1.1. Study of 3D sight distance.....	171
7.1.2. Effect of sight distance on highway safety.....	173
7.1.3. Evaluation of the 3D alignment coordination	175
7.2. Contributions	178
7.3. Future research.....	179
Chapter 8. REFERENCES	181
Appendix I. SELECTED HIGHWAYS.....	197
Appendix II. AVAILABLE SIGHT DISTANCE RESULTS	221
Appendix III. SIGHT-HIDDEN DIPS PARAMETERS	347

List of tables

Table 2.1. Driver's eye height and obstacle height values proposed by diverse highway standards in SSD analysis.	19
Table 2.2. Minimum offset required for horizontal curves on crests (FGSV, 2008a).....	29
Table 2.3. References to relevant sight-hidden dip parameters in literature.	42
Table 2.4. 85-th percentile values of the speed environment (McLean, 1981).....	46
Table 2.5. Interpretation of confidence and significance level in statistics (Pardillo, 2004).	54
Table 3.1. Main features of selected two-lane rural highway links.	91
Table 3.2. Summary of the surveying data of the selected highways.....	93
Table 4.1. Statistical facts of ASD values in highway M-221.	100
Table 4.2. Statistical facts of ASD values in highway M-325.	100
Table 4.3. Results of the paired samples comparison.....	101
Table 5.1. Simulation variables.	109
Table 5.2. Results of ASD and reliability analysis.	115
Table 5.3. Desired speed on the selected highway links.....	125
Table 5.4. Coefficients of friction (Richl and Sayed, 2006).....	126
Table 5.5. Comparison of ASD values among the three models.....	128
Table 5.6. Comparison of P_{nc} values among the three models under both profile assumptions.	129
Table 5.7. Comparison of empirical cumulative frequencies at design P_{nc} values.	131
Table 5.8. Theoretical safety level and consistency of P_{nc} values.....	133
Table 5.9. Coefficients and p-values of the negative binomial model.....	138
Table 5.10. Correlation matrix for coefficient estimates.	139
Table 6.1. Classification of crest – sag sequences on the selected highways.....	145
Table 6.2. Statistical facts of observed range values in hidden dips.....	147
Table 6.3. Statistical facts of the length of interim hidden sections in hidden dips.	147
Table 6.4. Statistical facts of the distance to the reemerged roadway in hidden dips.....	148
Table 6.5. Statistical facts of area in graph of the interim hidden sections.....	148
Table 6.6. Results of the paired samples comparison of hidden dip parameters.	148
Table 6.7. Statistical facts of observed range values in diving sections.....	149
Table 6.8. Statistical facts of the length of interim hidden sections in diving sections.	149
Table 6.9. Statistical facts of the distance to the reemerged roadway in diving sections.....	149
Table 6.10. Statistical facts of area in graph of the interim hidden diving sections.	150
Table 6.11. Results of the paired samples comparison of diving section parameters.	150
Table 6.12. Statistical facts of observed range values in jumping sections.	152
Table 6.13. Statistical facts of the length of interim hidden sections in jumping sections.....	152
Table 6.14. Statistical facts of the distance to the reemerged roadway in jumping sections. .	152
Table 6.15. Statistical facts of area in graph of the interim hidden jumping sections.....	153
Table 6.16. Results of the paired samples comparison of jumping section parameters	153
Table 6.17. Dependent variables considered on crest – sag sequences.....	155
Table 6.18. Description of the basic independent variables on crest – sag sequences.....	156
Table 6.19. Most significant variables affecting the probability of sight-hidden dips occurring on DTM01.....	157

Table 6.20. Most significant variables affecting the probability of sight-hidden dips occurring on DSM01.....	158
Table 6.21. Most significant variables affecting the range of hidden dips on DTM01.	159
Table 6.22. Most significant variables affecting the range of hidden dips on DSM01.	159
Table 6.23. Most significant variables affecting the length of interim hidden section on DTM01.....	160
Table 6.24. Most significant variables affecting the length of interim hidden section on DSM01.....	160
Table 6.25. Most significant variables affecting the distance to reemerged roadway on DTM01.....	161
Table 6.26. Most significant variables affecting the distance to reemerged roadway on DTM01.....	161
Table 6.27. Most significant variables affecting the area of hidden section in graph on DTM01.....	162
Table 6.28. Most significant variables affecting the area of hidden section in graph on DSM01.....	162
Table 6.29. Minimum offset required for spiraled horizontal curves on crests on motorways.	166
Table 6.30. Minimum offset required for non-spiraled horizontal curves on crests on motorways.	166

List of figures

Figure 1.1. Flowchart of the methodology followed in this thesis.	7
Figure 2.1. Comparison of tangential friction values given by different geometric design guides.	13
Figure 2.2. 2D modelling of ASD on horizontal curves longer than the ASD.	16
Figure 2.3. Graphical estimation of ASD on plans according to AASHTO (2011).	20
Figure 2.4. Stare areas at different speeds (Babkow, 1975).	23
Figure 2.5. a) The standard case of coordination between horizontal and vertical alignment; and b) a legitimate case of coordination where one phase is skipped (AASHTO, 2011).	26
Figure 2.6. Sequence of standard spatial elements (FGSV, 2008a).	27
Figure 2.7. Sequences of alignments on the vertical projection (FGSV, 1995b).	27
Figure 2.8. Shift from tangent to curve in plan hidden by a crest (VSS, 1991).	29
Figure 2.9. Shift from left turn to right turn in plan hidden by a crest (Ministerio de Fomento, 2000).	29
Figure 2.10. Sight-hidden dips a) on a tangent y b) on a horizontal curve (FGSV, 1995b).	30
Figure 2.11. Sight-hidden dips: a) horizontal sway or jump, and b) flutter (FGSV, 1995b).	30
Figure 2.12. Required distance to the end of the hidden section (VSS, 1991).	31
Figure 2.13. Comparison among visual fixation and PSD.	32
Figure 2.14. Perspective of a sag curve preceding a horizontal bend (Kraemer et al., 2009). ...	32
Figure 2.15. Perspective of a horizontal curve on a sag (Zimmermann, 2001).	33
Figure 2.16. Apparent roadway narrowing, a) horizontal and vertical projections and b) in perspective (VSS, 1991).	33
Figure 2.17. Sharp horizontal bend (VSS, 1991).	34
Figure 2.18. Sharp bend in the longitudinal profile (VSS, 1991).	34
Figure 2.19. Representation of a grade on a tangent (Kraemer et al., 2009).	35
Figure 2.20. Recreation of the highway perspective from the viewpoint of a driver by VESTRA software (Bark and Schmidt, 2010)	36
Figure 2.21. Geometric layout on a section where a sight-hidden dip exists (Easa, 1994a).	37
Figure 2.22. Roadway perspective depiction a) as designed and b) with null grade. (Zimmermann, 2001).	39
Figure 2.23. Critical hidden section according to the German standard for highway visualization (FGSV, 2008a).	41
Figure 2.24. Signposted long sight-hidden dip where, conversely, passing is not prohibited (Dixon, 2011).	42
Figure 2.25. Heavy-vehicle speed reduction/increase according to the Spanish standard (Ministerio de Fomento, 2016).	45
Figure 2.26. Comparison of 50-th percentile speed and standard deviation curves.	45
Figure 2.27. Possible transition speed profiles between successive curves (Castro et al., 2008b).	48
Figure 2.28. Variation of hazard index depending on traffic volume (Pardillo, 2004).	50
Figure 2.29. Accident rate as a function of the ASD (Krebs and Klöckner, 1977).	57
Figure 2.30. Basic components of a LSF.	61
Figure 3.1. 3D Sketch of line of sight retrieval in ArcScene.	70
Figure 3.2. ASD estimation through a line-of-sight loop.	70

Figure 3.3. Flowchart of sight distance study procedure.....	71
Figure 3.4. Sketch of airborne LiDAR survey.	73
Figure 3.5. Sketch of the point cloud layout obtained from a) airborne LiDAR and b) MMS.	74
Figure 3.6. Centerline drawn on an orthophoto from the IGN.	75
Figure 3.7. Azimuth graph from the polyline sketch on highway M-600.....	76
Figure 3.8. Determination of the relevant buffer area to build the DTM.....	77
Figure 3.9. Selection of the DTM cells overlapped with buffer.	77
Figure 3.10. Model builder workflow.....	78
Figure 3.11. Centerline and path feature classes at an intersection.	79
Figure 3.12. a) Cantilever sign, and b) representation of such a sign by a DSM derived from airborne LiDAR.....	80
Figure 3.13. Scheme of overhanging elements removal.....	81
Figure 3.14. Point of the roadway area and its vicinity zones.	82
Figure 3.15. Recreation of a bridge in the elevation model a) original DTM TIN; b) centerline profile before and after; c) roadway points considered for recreation, and d) processed DTM TIN.	83
Figure 3.16. a) MMS DSM with a tree overhanging the roadway, and b) the same elevation model after being processed.	84
Figure 3.17. Azimuth graph of the polyline and the deduced alignments on highway M-600...	86
Figure 3.18. Spiral geometric parameters.....	87
Figure 3.19. Example of profile reconstruction on highway M-104.	89
Figure 3.20. Example of vertical reverse curves adjustment.	90
Figure 3.21. Location of the selected highway links.	92
Figure 3.22. View of highway M-607 from a driver's perspective.	92
Figure 3.23. View of highway M-222 from a driver's perspective.	92
Figure 3.24. View of highway M-629 from the driver's perspective.....	93
Figure 3.25. View of motorway BAB-9 from a driver's perspective (Panoramio, 2015).....	94
Figure 4.1. Mound causing hidden section and visible roadway beyond in M-607.....	97
Figure 4.2. Sight distance graph comparing the outputs using DTM01 and DSM01 in M-607...	97
Figure 4.3. Items by the roadsides that reduce sight distance in M-221.....	98
Figure 4.4. Sight distance graph comparing outputs from a DTM and a MMS DSM in M-221...	99
Figure 4.5. View of highway M-325 in area with sparse vegetation.....	101
Figure 4.6. Comparison of ASD in area with sparse vegetation.....	102
Figure 4.7. View of highway M-325 in slightly wooded area.	102
Figure 4.8. Comparison of ASD in slightly wooded area.	103
Figure 4.9. View of highway M-325 in densely wooded area.	104
Figure 4.10. Comparison of ASD in densely wooded area.	104
Figure 5.1. Flowchart of the simulation process.	109
Figure 5.2. 50-th percentile speed profiles in a subsection of highway M-607.....	112
Figure 5.3. Numerical integration of SSD equation.....	113
Figure 5.4. Average ASD depending on the input heights.	114
Figure 5.5. Margin of compliance histogram.	114
Figure 5.6. Margin of compliance density map.....	115
Figure 5.7. P_{nc} after segmentation and vertical alignment.	116
Figure 5.8. Comparison of 2D horizontal ASD and curve radius in sample.....	118

Figure 5.9. ASD graph on a crest longer than ASD.	118
Figure 5.10. Lines of sight on a crest longer than ASD.	119
Figure 5.11. Parameters defining the line of sight on a crest shorter than ASD.	119
Figure 5.12. ASD and inverse ASD around a horizontal curve on highway M-325.	121
Figure 5.13. Speed profile of different acceleration models over distance.	123
Figure 5.14. Speed profile of different deceleration models over distance.	124
Figure 5.15. Empirical cumulative distributions of P_{nc} of the whole curve sample.	131
Figure 5.16. Empirical cumulative distributions of P_{nc} according to design speeds: a) v_d = 100 km/h; b) v_d = 80 km/h; c) v_d = 60 km/h; d) v_d = 40 km/h; e) off-standard.	132
Figure 5.17. Scatter plot of ASD and P_{nc} values.	134
Figure 5.18. Scatter plot of the initial speed and P_{nc} values.	134
Figure 5.19. Histogram of grade frequencies.	135
Figure 5.20. Cloud of points and interpolated surface using regression.	136
Figure 5.21. Contour plot of the effect of grade.	137
Figure 5.22. Relationship between predicted collisions and the P_{nc} with 95% confidence limits.	139
Figure 5.23. Consistent and non-consistent P_{nc} cumulative probability curves.	140
Figure 6.1. Identification of parameters characterizing sight-hidden dips in the sight distance graph.	146
Figure 6.2. Diving section on highway M-611.	151
Figure 6.3. Sight distance graphs displaying results of diving section on highway M-611 for a) DTM01 and b) DSM01.	151
Figure 6.4. Hidden dip with jumping reemerged section in highway M-222.	153
Figure 6.5. Sight distance graphs depicting results of jumping section on highway M-222 for a) DTM01 and b) DSM01.	154
Figure 6.6. Depiction of the vertical alignment parameters considered.	157
Figure 6.7. Alignment projections on hidden beginning of curve.	167
Figure 6.8. Visual discontinuity of the roadway caused by median barrier.	168
Figure 6.9. Sight-hidden dip with jumping.	168

List of abbreviations

AADT	Annual average daily traffic
AASHTO	American Association of State Highway and Transportation Officials
ASD	Available sight distance
CMF	Collision modification factor
CCR	Curvature change rate
DEM	Digital elevation model
DGT	<i>Dirección General de Tráfico</i> (Spanish Directorate General of Traffic)
DSM	Digital surface model
DTM	Digital terrain model
FGSV	<i>Forschungsgesellschaft für Straßen- und Verkehrswesen</i> (German Society for Road and Traffic Research)
FORM	First order reliability method
GLM	Generalized linear model
GNSS	Global Navigation Satellite System
H ViSt	<i>Hinweise zur Visualisierung von Entwürfen für außerörtliche Straßen</i> (Recommendations for visualizing the design of rural roads)
IGN	<i>Instituto Geográfico Nacional</i> (Spanish National Geographic Institute)
IMU	Inertial measurement unit
LiDAR	Light detection and ranging
LSF	Limit state function
MMS	Mobile mapping system
MSSE	Motorway standard spatial element
MTN	<i>Mapa Topográfico Nacional</i> (Spanish National Topographic Map)
MUTCD	Manual on Uniform Traffic Control Devices
MVFOSM	Mean-value first-order second-moment
PSD	Passing sight distance
RAL	<i>Richtlinien für die Anlage von Landstraßen</i> (Standards for construction of rural roads)
RMS	Root mean square
RRR	Resurfacing, restoration and rehabilitation
SETRA	<i>Service d'études sur les transports, les routes et leurs aménagements</i> (Department of transportation, roads and facilities)
SORM	Second order reliability method
SPF	Safety performance function
SSD	Stopping sight distance
SSE	Standard spatial element
TIN	Triangular irregular network
VBA	Visual basic for applications
WHO	World Health Organization

Chapter 1

INTRODUCTION

1.1. Background

An estimated 1.2 million people are killed in the world road system and as many as 50 million are injured each year according to the World Health Organization (WHO, 2015), a figure that is regarded as staggering high. While mobility in the world is expected to increase in the years to come, societies face the challenge of reducing morbidity on highways. Besides the loss of human lives, road collisions are the cause of huge economic losses. These losses hamper the economic development and competitiveness of countries.

The factors determining driving performance are many and varied. Those related to the design of highway facilities include geometrics, access control, pavement condition, and safety devices. Highway design comprises multiple areas, among which the geometric design deserves the utmost attention. The importance of design also stems from aspects as diverse as highway capacity, users' comfort and sustainability. The vehicle maintenance, human factors, environmental conditions and legal issues are also relevant aspects in driving performance.

Road accidents are produced by a complex mix of the factors listed above. Transportation engineers and designers are getting away with safety issues by attributing the causes of accidents to road users. Nonetheless, improvements in design criteria and operation policies can render a decisive contribution to highway safety.

Whereas highway planning, construction, operation and maintenance require large capital investments, accident risk constitutes an unavoidable liability to highway operation that must be minimized. Thus it is necessary to intensify the search for new tools and techniques which enable designers to check the design or the current status of facilities, clear up the causes of accidents and learn from the experience gained delivering enhanced policies and standards. Such an experience will result in more consistent, sustainable and safer design criteria for new highways, as well as enhanced upgrade policies and countermeasures on in-service facilities in order to eventually fix the road system.

Sight distance is, in conjunction with speed, the most important geometric design control criteria. The safe and efficient operation of highways largely depends upon the provision of adequate sight conditions. Although the linear dimension prevails, roads are three dimensional (3D) in nature, their geometry describing a 3D strip in the space. Therefore, sight distance and

its effects on driving performance must be considered in accordance with this 3D nature in order to create a suitable driving space.

1.2. Motivation

Highway safety concerns are currently of utmost importance. The optimization of design can render a decisive contribution to increased highway safety.

New surveying technologies available, namely Light Detection And Ranging (LiDAR), permit massive and precise geospatial data collection of a highway and its roadsides. After an appropriate treatment, the information retrieved may be apt for 3D sight distance modelling. Prerequisite to conducting sight distance analysis is that an efficient computer tool for the sight distance calculation is available. Increased computational capabilities permit nowadays the treatment and analysis of large datasets and the use of more complex techniques that retrieve more precise results. Particularly, Geographic information systems (GIS) combine the rapid computational, data retrieval and manipulation as well as excellent analysis capabilities. The use of these techniques enables a more accurate and realistic analysis of sight distance conditions unavailable up to now.

Highway geometric design has traditionally contemplated design variables as deterministic whereas the variables involved in highway design are stochastic in nature. A switch from the traditional deterministic approach to a new probabilistic approach in highway design has been advocated through the reliability analysis technique. This technique enables a more precise estimation of failure risk in design. The evaluation of failure risk can act as a surrogate safety assessment. Reliability theory holds an important advantage over road collision analysis for the purpose of safety assessment as it provides less-costly insight into the failure mechanisms that lead to accidents.

Given 3D nature of sight distance, its study and analysis must cope with well-defined spatial inputs that represent these facilities. The 3D outcome of highway design is the reality that the road user will have to deal with. The principles of 3D alignment coordination are mostly descriptive. Although different quantitative techniques have been developed to analyze the spatial alignment coordination, they either showed little effectiveness or did not meet success owing to the difficulty of implementation.

As nowadays accident rates are higher on two-lane highways than on motorways, concerns are increasingly being transferred to two-lane highways. Thus, the study of geometric design of two-lane rural highways using sight distance analysis and on the effect of sight distance on highway safety deserves special attention. Nonetheless, the enhancement of high capacity facilities should be pursued as well by assessing the spatial alignment of motorways.

1.3. Statement of the research problem

The study of sight distance on highways draws its importance from the impact it has on geometric design, operation and traffic safety. A procedure capable of studying sight distance on in-service highways and of delivering reliable sight distance measurements in 3D is still lacking. Such a comprehensive 3D sight distance methodology of analysis on in-service highways

needs therefore to be devised, including appropriate data treatment to produce the required inputs.

Different standards around the world provide deterministic criteria to broach the geometric design of highways whereas the systems modelled are stochastic by nature. Sight distance is one of these control criteria. This approach has two drawbacks. First, the selection of the design values is not founded on definitive safety levels. Second, there is little knowledge on the safety implications of deviating from the geometric design standards. Moreover, it is not infrequent that projects do not adhere to those standards. This fact may also lead to inconsistent uncertainty levels within the design of a given highway. Therefore, a methodology to estimate such risks on in-service highways is necessary.

The impact that inadequate sight distance has on highway safety has been extensively studied. Nonetheless, the methodologies utilized so far for the estimation of sight distance in this kind of studies might not have brought reliable results. Moreover, by incorporating sight-distance-related reliability measurements in accident prediction models, it will be possible to link collision risk and sight distance issues according to an appropriate methodology. Consequently, a statistical analysis has to be performed using unbiased independent variables to provide insight on the effect of sight distance on accident occurrence.

Finally, the 3D appearance of a highway is what the driver perceives and defines the ease of operation and, at the end of the day, safety. Detached design of horizontal alignment, vertical alignment and cross section may generate a solution lacking a proper appearance. Most 3D alignment coordination criteria provide qualitative recommendations to prevent shortcomings in the spatial alignment. In an effort to improve facilities, useful quantitative assessment criteria must be elaborated to aid designers.

1.4. Objectives

Based on the facts above explained about sight distance on highways and safety, the main objective of this doctoral thesis is to develop a methodology for the study of 3D sight distance on highways, analyzing its effects on safety and 3D alignment coordination. From the main objective, the following specific goals were set within the scope of this thesis:

1. To review the currently available techniques for the estimation and analysis of highway sight distance, alignment coordination and their relation to highway safety. As a result, an application must be selected for the sight distance calculation.
2. To define and validate a procedure to create a useful cartographic database, including data selection and treatment, for the sight distance calculation and the subsequent analysis of results. Such a cartographic database must comprise the inputs required by the selected calculation tool.
3. To study the effect of the elevation model type employed to sketch the roadway and the roadsides on the sight distance outcome. The elevation models to be studied include a terrain and a surface model derived from airborne LiDAR, and surface model derived from terrestrial LiDAR.
4. To analyze the effect of available sight distance on highway safety from a stochastic approach through reliability theory, focusing both on full-segment analysis and on

restricted sight distance locations. The available sight distance results to be utilized come from different sight distance modelling approaches.

5. To model accident frequency in curves through generalized linear models based on the aforementioned reliability results.
6. To analyze the effect of the elevation model type on the characterization of sight-hidden dips. The elevation models to be studied include a terrain model and a surface model.
7. To analyze the relationships between the variables defining the 3D alignment coordination and the occurrence of sight-hidden dips and their parameters.
8. To study the shortcomings in the spatial alignment of motorways and the conditions under which they are produced.
9. To propose design recommendations in order to avoid shortcomings in the spatial alignment of highways.

1.5. Methodology

The methodology followed in the elaboration of this doctoral thesis comprises several phases. To achieve the objectives defined in the previous section, the sequence of stages followed is outlined in the following subsections.

1.5.1. Elaboration of literature review

Prior to address research on the topic of this thesis, a review of the current status of knowledge and techniques is necessary. The critical analysis of the content of such a review allows the identification of weaknesses and inconsistencies in order to propose improvements of the current techniques and make advances in knowledge.

To tackle the study of highway sight distance, alignment coordination and their effect on traffic safety, this thesis covers a wide number of fields owing to the different techniques utilized:

- Sight distance: this is the core aspect analyzed in this thesis. The review covers required sight distance for different driving maneuvers and sight distance estimation techniques.
- Sight distance and driving behavior: in this part, the advances in knowledge about highway visual appearance to drivers in order to meet their expectations is reviewed. This is an introductory aspect for the analysis of 3D alignment coordination.
- 3D alignment coordination: this part includes a description of the main spatial alignment shortcomings on highways, as well as the current techniques to prevent them from existing, or to detect and address them otherwise.
- Operating speed: the review of operating speed models is necessary to produce theoretical speed profiles for the evaluation of sight distance requirements. It goes over speed patterns and relations to alignment features.
- Highway safety: in this part, safety indices and statistical tools utilized in safety analysis are described. Also, the foremost findings about the relationship between sight distance and safety are reviewed.
- Reliability theory: this technique, which was selected for the evaluation of sight distance requirements, facilitates the analysis of engineering systems from a probabilistic approach.

- Alignment data extraction: as the sketch of operating speed profiles is based on geometrics, the alignment of the highways to be selected needs to be fully determined. It is also needed for the characterization of alignment coordination.

1.5.2. Methods, materials and procedure development

Based on the review, this thesis utilizes available tools and techniques in the assignments where it is considered that they can provide good performance and reasonably unbiased results. These tools refer to a GIS-based application for sight distance estimation, RT for reliability-based analysis of sight distance restrictions in curves, and CARD/1 for the evaluation of the 3D alignment of motorways.

The selection of case studies of two-lane rural highways was made after a visual inspection of highways on *Google Street View*® in search of sections where sight distance issues are present. The elevation models for the case studies come from either airborne LiDAR or terrestrial LiDAR. Whilst the first ones were available for almost the entire territory of Spain, the second ones were collected after the selection of case studies. In the case of motorways, the case study was chosen according to data availability.

Furthermore, own developments of the author are presented, they being devised to resolve issues in aspects where no tool is available or reliable. Given the nature of data, the studies to be carried out and their corresponding inputs motivated the development of additional procedures. Data preparation enabled the creation of a suitable cartographic database. These developments for data preparation and analysis include:

- MATLAB scripts to processing the elevation models and terrestrial LiDAR data.
- Several scripts developed in Visual basic for applications (VBA) to aid in the deduction of the theoretical alignment in plan and profile and the computation of 3D alignment coordination variables.
- MATLAB scripts and functions for the reliability analysis through a Monte Carlo simulation.
- MATLAB script for automating the reliability analysis computation in RT.

1.5.3. Pilot studies

First, pilot studies were formulated in reduced sets of case studies to test the feasibility of the methodology proposed. In some aspects, they delivered relevant results and in others interesting conclusions were found and applied to the subsequent full study.

In the sight distance pilot study, the outcomes of the 3D method using three different elevation models and from different sources were compared in two of the selected highways. The features of these two highways and their respective roadsides (e.g. sight restrictions caused by vegetation, metallic barriers and the roadway itself) made them suitable for this pilot study. Statistical inference tests were applied to check the hypothesis of whether the sight distance samples come from the same distribution.

The reliability study using a Monte Carlo method was applied to one of the selected highways. That one was chosen because its design matches the higher standards so the implementation of the simulation could be done at ease as detailed in section 5.2.

1.5.4. Complete case study

1.5.4.1. Sight distance computation

According to the findings in the pilot study, sight distance was computed in all the selected two-lane highways using the elevation models from airborne LiDAR.

1.5.4.2. Reliability analysis on horizontal curves

Sight distance results were retrieved for the application of reliability theory to stopping sight distance in curves of the selected highway links. The findings in the pilot study were taken into account for the variables to be considered. In addition, 2D sight distance values were incorporated in this analysis. Furthermore, criteria for the curve selection and classification were devised. For the analysis of results, statistical inference tests were applied to check whether the safety implications of the sight distance modelling approaches are significantly different. The different variations also included the consideration of the grade through the stopping maneuver.

Then, the results of such reliability analysis were incorporated in statistical models to evaluate the possible effect that insufficient sight distance has on real accident occurrence.

1.5.4.3. 3D alignment coordination

On the one hand, the study of 3D alignment coordination on two-lane highways focused on sight-hidden dips because this spatial alignment shortcoming is considered relevant for safety and can be studied at ease through sight distance analysis. The effect of sight distance modelling inputs on sight-hidden dips is first researched. Then, the 3D alignment coordination variables that are more likely to produce such design flaws are identified.

On the other hand, the study of spatial alignment coordination of motorways is carried out. Several shortcomings in the perspective, related to sight distance and interim hidden sections caused by an inadequate spatial superposition of alignments, were analyzed. Recommendations are provided on the basis of the analysis

1.5.5. Conclusions and future lines of research

After the analysis of results, the subsequent conclusions were drafted, including the contributions of this research. Finally, future lines of research are proposed from the results of this thesis.

This methodology is outlined in the flowchart shown in Figure 1.1

Figure 1.1. Flowchart of the methodology followed in this thesis.

Appendices II and III are available in the digital version of this document only. The printed version of this thesis includes a CD attached where the full digital version of this document, including Appendices II and III, can be found.

Chapter 2

LITERATURE REVIEW

2.1. Introduction

The importance of geometric design in safe highway operation is acknowledged by highway design guides. The alignment is, moreover, the first aspect to be considered when designing a highway. The process of highway geometric design involves the full comprehensive spatial alignment and dimensioning of all the roadway and roadside features aiming at providing safe, comfortable and aesthetically pleasant driving conditions in a cost-effective manner. Because highway construction and management require large capital investments, the design should be optimized looking for a trade off with safety.

Sight distance is a major design element utilized as a dimensioning criterion of the alignment elements. Moreover, sight distance analysis may be employed in already-built highways to detect possible deficiencies in the geometric design or in the roadside maintenance, which may have safety implications. Hence the importance of developing a reliable methodology for estimating available sight distance.

Despite the 3D nature of the road, the usual way design engineers tackle highway geometry begins with the isolated consideration of the horizontal alignment, the vertical alignment and the cross section. However, satisfactory results may not be achieved unless these three features are suitably arranged. During the design stage, designers have to keep in mind that the driver does not have a view of the roadway as the engineer has on the project drawings. In highways where good coordination between horizontal and vertical alignment has been achieved, drivers are aided to perceive the spatial elements of the alignment properly. Hence design engineers should create an easily understandable alignment to drivers, so that they are provided with guiding hints whenever possible. This way, the driver can foresee the due path, leading to safer driving.

2.2. Sight distance

2.2.1. General considerations

The need for drivers to see ahead is of the utmost importance in the safe and effective operation of their vehicles on a highway. Thus sight distance is one of the paramount parameters of highway geometric design (AASHTO, 2011). Motor vehicles are subject to the control of drivers on the roadway, whose speed and behavior may vary greatly depending on their experience and

ability. A driver travelling along a highway section needs to perceive information of the driving environment that facilitates such a proper operation. The roadway and its roadsides, the posted signs and road markings as well as the presence of other users are fundamental components of the driving environment to be perceived.

2.2.2. Required sight distance

Designers should ensure satisfactory enough sight distance conditions so that drivers can control the operation of their vehicles, interact with traffic conditions and environmental features to perform driving maneuvers safely.

Standards define the required sight distance for the maneuvers inherent in driving such as emergency stopping, passing, decision making or merging the traffic flow. A certain minimum sight distance would permit most drivers to make appropriate decisions to perform each maneuver comfortably and without compromising safety. Literature provides designers with models to estimate such distances, which fundamentally depend on the speed of the vehicles involved, among other factors.

2.2.2.1. Stopping sight distance

Stopping sight distance (SSD) is normally defined as the stretch travelled by a vehicle during a forced stopping maneuver, measured from where the hazard is first perceived to the point where the vehicle stops completely (Ministerio de Fomento, 2016)¹. This maneuver is modelled as the sum of two terms: the distance travelled during the time the driver needs to perceive the hazards and react to it by applying the brakes; and the braking distance itself.

According to AASHTO (2011), the SSD is evaluated in the general case according to the following expression:

$$SSD = 0.278vt_p + \frac{v^2}{254\left(\frac{a}{9.81} \pm G\right)} \quad (2.1)$$

where:

- SSD = stopping sight distance, m;
- v = design speed, km/h;
- t_p = perception-reaction time, 2.5 s;
- a = deceleration rate, 3.4 m/s²;
- G = grade, m/m.

The formula provided by the Spanish standard (Ministerio de Fomento, 2016), which is substantially similar to that of the AASHTO, the acceleration term is replaced by the tabulated values of tangential friction factor f_t that depend on the speed v . The f_t values range from 0.432 (for 40 km/h) to 0.249 (for 150 km/h).

¹ In what follows, unless otherwise stated, the citations of the current Spanish geometric design standard (Ministerio de Fomento, 2016) apply also to the previous one (Ministerio de Fomento, 2000).

$$SSD = \frac{vt_p}{3.6} + \frac{v^2}{254(f_t + G)} \quad (2.2)$$

where:

- SSD = stopping sight distance, m;
- v = design speed, km/h;
- f_t = coefficient of tangential friction between wheel and pavement;
- G = grade, m/m;
- t_p = perception-reaction time, 2 s.

In standards and guides, the skid resistance is used to define the maximum permissible tangential friction factor by the 90-th or 95-th percentile distribution curve (Lamm et al., 1995).

Other geometric design guides offer more complete (and complex) expressions. On the one hand, they leave a wide margin of discretion to designers, yet those designers, on the other hand, will have to justify their input values. Examples include the Italian guides for geometric design of highways (Ministero delle Infrastrutture, 2001), which evaluates SSD by means of the following integral within the braking distance addend:

$$SSD = \frac{v_0 \cdot t_p}{3.6} - \frac{1}{3.6^2} \int_{v_0}^{v_1} \frac{v}{g \left[f_t(v) \pm \frac{G}{100} \right] + \frac{F_a(v)}{m} + r_0(v)} dv \quad (2.3)$$

where:

- SSD = stopping sight distance, m;
- v_0 = initial vehicle speed, km/h;
- v_1 = final vehicle speed (0 in case of stopping), km/h;
- G = grade, %;
- t_p = perception-reaction time, s;
- g = gravitational acceleration, 9.81 m/s²;
- F_a = aerodynamic friction, N;
- m = vehicle mass, kg;
- f_t = threshold of the coefficient of tangential friction between wheel and pavement;
- r_0 = unitary resistance to rolling (negligible), N/kg.

Here, perception-reaction time is considered variable depending on the speed (in km/h) and according to the following expression:

$$t_p = 2.8 - 0.01 \cdot v \quad (2.4)$$

The aerodynamic friction factor is evaluated as follows:

$$F_a = \frac{\rho \cdot C_x \cdot S \cdot v^2}{2 \cdot 3.6} \quad (2.5)$$

where:

- C_x = aerodynamic-friction coefficient;
- S = reference area of master section, m²;
- ρ = air density under standard conditions, kg/m³;

However, the document presents two lists of tabulated values for the tangential friction: one for motorways and another for other highways.

The former German geometric design standard (FGSV, 1995b), much like the Italian counterparts, proposed a detailed SSD mathematical model, which is given by the following expression:

$$SSD = \frac{v_0}{3.6} t_p + \frac{1}{3.6^2 \cdot g} \int_{v_1}^{v_2} \frac{v}{f_t(v) + \frac{G}{100} + \frac{F_a}{W}} dv \quad (2.6)$$

where:

- SSD = stopping sight distance, m;
- v_2 = initial vehicle speed, km/h;
- v_1 = final vehicle speed (0 in case of stopping), km/h;
- t_p = perception-reaction time, 2 s;
- f_t = unitary tangential friction, which is estimated according to:

$$f_t = 0.241 \left(\frac{v}{100} \right)^2 - 0.712 \left(\frac{v}{100} \right) + 0.708 \quad (2.7)$$

- G = grade, %;
- F_a = vehicle aerodynamic friction, N.
- W = vehicle weight, N.

The ratio between the two latter ones is estimated as a function of the square of speed:

$$\frac{F_a}{W} = 3.27 \cdot 10^{-4} \cdot \left(\frac{v}{3.6} \right)^2 \quad (2.8)$$

Most models assume the higher the tangential friction is, the shorter the SSD will be. According to this, the tangential friction of the SSD models can be compared in Figure 2.1. It can be observed that, whereas the AASHTO (2011) standard provides the most conservative values for the lower speeds, the German standard (FGSV, 1995b) is posited further on the safe side at higher speeds.

Figure 2.1. Comparison of tangential friction values given by different geometric design guides.

The distribution of t_p , which depends solely on the driver's abilities, has been studied by many researchers. Lerner (1995) affirmed that t_p comes from a lognormal distribution with mean 1.5 s and standard deviation 0.4 s. While the Spanish standard (Ministerio de Fomento, 2016) and the German standard for rural roads (FGSV, 1995b) propose 2 s as t_p , the American Association of State Highway and Transportation Officials (AASHTO, 2011) suggests a greater lapse of 2.5 s. Green (2000) studied the factors involved in human braking performance, finding that drivers can respond to an unexpected event in about 1.5 s on average.

The current German highway design guide (FGSV, 2012), applicable to two-lane rural highways, abandons the two-stage model. Roos et al. (2005) maintain that the two-stage model is obsolete since it was devised many years ago for highways whose design speed was between 60 and 80 km/h and extrapolated with no guarantee of validity at higher speeds. The latest technologies in automobile brakes have, furthermore, significantly enhanced the braking performance since then. The new model proposed, which assumes constant deceleration rate from start to end, was developed throughout an empirical study on pavement surfaces varying the adherence conditions, with diverse vehicles and different tire wear. The calibration of the model was aided by artificial neural networks. It was finally observed that t_p diminishes as the initial speed increases. The study also concluded that the influence of wet pavement is not as significant as it was thought. No geometric design standard has been found following this trend. AASHTO, Canada, France, Italy, Poland or Spain, among others, still consider the model based on perception plus reaction.

2.2.2.2. Decision sight distance

AASHTO (2011) introduced a few years ago the concept of decision sight distance to describe the driver demand of sight distance in complex maneuvers at critical locations, i.e. while approaching an intersection, approaching a toll plaza, etc. Such a distance would enable the driver to adapt their speed and driving path without compromising comfort nor safety. Models to estimate the drivers need were theorized for different speeds, scenarios and environments (rural, suburban or urban), yielding five different avoidance maneuvers. For each avoidance maneuver, the pre-maneuver time varies, ranges from 3 to 14.5 s. In any case, decision sight distances are always noticeably longer than the SSD. The current Spanish standard (Ministerio de Fomento, 2016) also incorporated decision sight distance. The values proposed match the distance travelled during 10 s at the design speed.

2.2.2.3. Passing sight distance

Passing sight distance (PSD) is defined as that required by a vehicle to move ahead of another vehicle travelling at a slower speed in presence of opposing traffic (Ministerio de Fomento, 2016). These maneuvers will be accomplished safely when the passing driver is able to see a long-enough section of highway clear of traffic without cutting off the just passed vehicle in the event of an opposing vehicle appearing during the maneuver. When appropriate, the driver may abort the maneuver and return to the right lane without completing the pass if they consider the opposing traffic is too close when the maneuver is not fully completed (AASHTO, 2011). The minimum PSD values exhibited in the AASHTO guide are based on those presented in the Manual on Uniform Traffic Control Devices (MUTCD) (FHWA, 2009).

Generally speaking, a standard passing maneuver successfully accomplished comprises two phases associated each to a distance travelled. In the first one, the rear vehicle encroaches the opposing lane until it moves lane in parallel to the passed one. Next, the passing vehicle returns to the proper lane. Under these assumptions, the PSD can be computed as the sum of four distances (El-Bassiouni and Sayed, 2010):

- The distance the passing vehicle travels while accelerating to the point of encroachment on the left lane.
- The length of roadway that is travelled by the passing vehicle while it occupies the opposing lane.
- The clearance distance between the passing vehicle and the opposing vehicle when the passing vehicle returns to the right lane.
- The distance travelled by an opposing vehicle at the second phase.

Hassan et al. (1996b) reported inconsistencies in former passing models that refuse to assume the opportunities of aborting the maneuver before reaching a point of no return. Those former models led to exaggerated PSD requirements. Moreover, research has verified the current PSD values of AASHTO match field observations (Harwood et al., 2008). Recent research has, nonetheless, continued considering the four-phase model to study passing issues (El-Bassiouni and Sayed, 2010; Llorca et al., 2014b).

Similarly, the former German highway standard RAS-L (FGSV, 1995b) proposed a detailed model for the determination of PSD where it is assumed that the passing vehicle exceeds the legal speed limit. Instead, the new German standard RAS (FGSV, 2012), although somewhat based on the former model, currently provides single values in order not to promote exceeding the maximum legal speed. The current Spanish standards (Ministerio de Fomento, 2016; MOPU, 1987) set design criteria for passing zones based on sight distance at the beginning at the end and the minimum length of the section. Conversely, the previous geometric design standard (Ministerio de Fomento, 2000) offered a completely different criterion that mismatched the Spanish road marking standard (MOPU, 1987), a fact that was repeatedly reported (Llorca et al., 2014a; Moreno et al., 2016).

Other techniques have been used to characterize passing maneuvers. For instance, Moreno et al. (2015) calibrated a micro simulation model to devise design criteria for minimum passing zone lengths. El Khoury and Hobeika (2007) researched the risk associated with some current PSD models also using micro simulation.

2.2.3. Available sight distance

Available sight distance (ASD) is defined as the section of roadway between the driver and the farthest object on the roadway that can be seen by this driver without the line of sight being interrupted by the terrain, nearby elements or the road itself. Under normal visibility conditions, the driver's line of sight may be obstructed not only by the highway itself but also by lateral obstacles such as cut-side slopes, topographic features, fences, hedges, wooded areas, buildings or even roadside features. Therefore the limitation of lateral clearances in bends or the road surface in crest vertical curves determines the ASD. AASHTO (2011) encourages considering sight distance in the preliminary stages of geometric design, when both the horizontal and vertical alignment are still subject to adjustment.

2.2.3.1. Available sight distance estimation

As it has already been pointed out, the consideration of the ASD on highways is of great importance at the design stage. It contributes by providing safety criteria not only to arrange the alignments appropriately but also to devise the vertical signaling layout and pavement markings. Moreover, the ASD estimation is also necessary during the service life of facilities, for example to elaborate highway inventories, to assess the existing signs and markings, or to perform safety audits. To broach this issue, several methods are available. They can be classified into 2D methods and 3D methods, which are discussed in detail below.

2.2.3.1.1. Two-dimensional estimation methods

On each projection, standards make criteria available to arrange the alignments in a proper layout of in relation to the required sight distance. Bends are assumed to determine the ASD on the horizontal projection and vertical curves do so in profile. Therefore, separate methods have been set to estimate the ASD on the horizontal and the vertical alignment.

a) Horizontal projection

The required clearance by the inner roadside of a curve that is longer than the ASD is deduced through geometric relations and is given by numerous standards (AASHTO, 2011; Ministerio de Fomento, 2016; VSS, 1991). The Spanish standard (Ministerio de Fomento, 2016) provides the following equation to compute the lateral clearance needed to provide a given ASD in horizontal curves (Figure 2.2):

$$C = R^* - (R^* + b) \cdot \cos\left(\frac{31.83 \cdot ASD}{R + b}\right) \quad (2.9)$$

where:

- C = minimum horizontal offset from the sight obstruction to the inner roadway edge, m.
- R^* = radius of the inner roadway edge, m.
- b = offset between the driver and the inner roadway edge, m.
- ASD = available sight distance, m.

Figure 2.2. 2D modelling of ASD on horizontal curves longer than the ASD.

AASHTO (2011) offers a very similar model with an equation in which some of the parameters vary slightly as the lane width is disregarded in this case.

$$C = R \cdot \left[1 - \cos\left(\frac{28.65 \cdot ASD}{R}\right) \right] \quad (2.10)$$

where:

- C = horizontal sightline offset from the obstruction to the vehicle path, m.

R = radius of curve, m.

ASD = available sight distance, m.

Equations (2.4) and (2.5) provide the exact clearance required in the case where the length of the curve is longer than the ASD, it being overestimated otherwise. Lovell et al. (2001a) provided a method to determine the clear areas in curves contemplating possible adjacent spirals and regardless of the curve length. Moreover, Lovell and Iida (2003) set the formulation to determine the exact clear zone envelope. The mathematical expressions given by guides and standards can be reformulated to estimate the ASD if it is cleared.

A necessary consideration on the horizontal projection to take account of is the position of both driver's eye and target in the highway cross section. The point of view and the target are usually assumed to follow a line parallel either to the highway centerline or the roadway edge. For example, the current Spanish standard sets the driver and target path on the own travelling lane at an offset of 1.5 m from its left edge to evaluate the SSD (Ministerio de Fomento, 2016), while the former one (Ministerio de Fomento, 2000) indicated that the observer and target path must be parallel to the outer lane edge at an offset of 1.5 m, both being on the centerline itself to evaluate the PSD. In the case of the French standard (SETRA, 1994) the path is set at 2 m from the right border of the roadway. The German standard for rural roads (FGSV, 2012) establishes such a path on the center of the own lane up to a distance of 400 m for the SSD evaluation. Beyond this value, the target is placed on the center of the opposing lane. Similarly, the Italian standard (Ministero delle Infrastrutture e dei Trasporti, 2001) sets the center of the own lane as observer and target location for SSD analysis while the target is moved to the center of the opposing lane in PSD analysis. On the contrary, AASHTO (2011) recommends evaluating the ASD on the road centerline.

Lovell (1999) devised an algorithm to compute the ASD on the horizontal projection given lateral clearance. The method was later enhanced to cover more complex conditions (Lovell et al., 2001b).

b) Vertical projection

As occurred on the horizontal projection, the curved elements, namely crests and sags are the segments that limit the ASD in profile. Whereas the crests are always assumed to obstruct sight distance, sag curves restrain the ASD at nighttime only. Exceptionally, the presence of overpasses could interfere with ASD (Easa, 2009). The dimensioning of sag curves is mainly governed by a provision of sufficient nighttime sight distance, which depends upon the upward divergence of the vehicle light beam. Crests and sags are mathematically defined as parabolas. The ASD on crest vertical curves is calculated as expressed in Equation (2.11), while in sag curves it is calculated according to Equation (2.12). A sag vertical curve, should be long enough so that the light beam distance is approximately the same as the required SSD. Accordingly, it is appropriate to use SSD for different design speeds as the value of S in the equations below.

$$K_v = \frac{SSD^2}{2(\sqrt{h_1} + \sqrt{h_2})^2} \quad (2.11)$$

$$K_V = \frac{SSD^2}{2(h_h - h_2 + ASD \cdot \tan \alpha)} \quad (2.12)$$

where:

- K_V = radius of the vertical curve, m.
- h_1 = driver's eye height, m.
- h_2 = target height, m.
- h_h = headlight height, m.
- α = upward divergence of the light beam from the longitudinal axis of the vehicle, 1°.
- SSD = required SSD, m.

The AASHTO (2011) approach to determine the ASD on vertical crest curves, unlike the Spanish model, considers apart the case in which the parabolic curve is longer than the required SSD.

$$L = \frac{\theta \cdot ASD^2}{100(\sqrt{2 \cdot h_1} + \sqrt{2 \cdot h_2})^2} \quad \text{if } S < L \quad (2.13)$$

$$L = 2 \cdot ASD - \frac{200(\sqrt{h_1} + \sqrt{h_2})}{\theta} \quad \text{if } S > L \quad (2.14)$$

where:

- L = length of vertical curve, m.
- θ = algebraic difference in grades, percent.
- ASD = available sight distance, m.
- h_1 = driver's eye height, m.
- h_2 = target height, m.

The same distinction applies to sag vertical curves, where available sight distance can be derived from the following expressions:

$$L = \frac{\theta \cdot ASD^2}{200[0.6 + ASD \cdot \tan(\alpha)]^2} \quad \text{if } S < L \quad (2.15)$$

$$L = 2 \cdot ASD - \frac{200(0.6 + ASD \cdot \tan(\alpha))^2}{\theta} \quad \text{if } S > L \quad (2.16)$$

Where:

- L = length of vertical curve, m.
- θ = algebraic difference in grades, percent.
- S = available sight distance, m.
- α = upward divergence of the light beam from the longitudinal axis of the vehicle, 1°.

Other particular cases of 2D sight distance models in profile have been analytically contemplated. Sight distance on sight-hidden dips is reviewed in detail in section 2.4.3.2.1 (Easa,

1994a). Sight distance on sag vertical curves with overpasses was also studied in 2D. Easa (1992) analyzed the case of a sag curve with a non-centered overpass, providing close form solutions of the minimum ASD. Some years later, Easa added the overpass width to the model and provided an algorithm to optimize the design by maximizing the ASD at the underpassing road (2009).

For measuring the distance along a roadway throughout which and object is visible to the driver, two fundamental parameters must first be considered. First, the height of the driver's eye above the road surface. Although this parameter is subject to a great variability (mainly due to the variety of drivers, vehicle models and vehicle types) standards of different countries propose fixed values meant to be on the safe side (Table 2.1). In this respect, Fitzpatrick et al. (1998) characterized the distribution of driver's eye height in the USA whereas Capaldo (2012) did so in Italy. Furthermore, some countries describe on their standards sight distance estimation procedures for trucks (AASHTO, 2011; Austroads, 2003; Department of Transport, 1993). Actually, according to a report by Glennon (1987), certain assumptions made by sight distance models do not lead to an accurate output. The higher driver's eye height of truck drivers does not necessarily compensate the greater mass of the vehicle when braking. A greater tangential friction is demanded when braking at bends due to the centripetal acceleration component. In addition, hindrances at the inner roadside may be higher than the truck driver's eye.

On the other hand, it is necessary to define the target height, which also varies depending on the standard considered. It differs in each guide depending on whether the ASD estimation deals with stopping or passing maneuvers. The target height values shown in Table 2.1 correspond to SSD analysis. Normally standards suggest the consideration of the same height value for the observer and the target than for the observer when examining PSD.

Table 2.1. Driver's eye height and obstacle height values proposed by diverse highway standards in SSD analysis.

Standard	Driver's eye height [m]	Target height [m]
AASHTO (2011)	1.08	0.60
ARP, France (SETRA, 1994)	1.00	0.35
Austroads (2003)	1.15	0.20
Canada (TAC, 1999)	1.05	0.38
Department for Transport, U.K. (1993)	1.00	0.26
H ViSt, Germany (FGSV, 2008a)	1.10	0.00
Norma 3.1-IC (M. Fomento, 2000)	1.10	0.20
Norma 3.1-IC (M. Fomento, 2016)	1.10	0.50
NFGCS, Italy (M. Infrastrutture, 2001)	1.10	0.20
RAL, Germany (FGSV, 2012)	1.00	0.10
VSS 640, Switzerland (VSS, 1991)	1.00	0.15
VU94, Sweden (SRNA, 1994)	1.10	0.20

c) Both projections detached

The 2D ASD estimation procedures consist of line-of-sight envelopes over the vehicle path. According to the AASHTO guide (2011), these lines could be sketched graphically as exhibited in Figure 2.3. Independent examinations are performed on each alignment projection for each direction of travelling. To elaborate the sight distance record, the minimum value is retrieved for each direction.

Figure 2.3. Graphical estimation of ASD on plans according to AASHTO (2011).

Nehate and Rys (2006) proposed an automated method to examine the ASD modelling the highway geometrics by means of a global position system (GPS) track separately on each projection. Also, current road inventories aim at providing an estimate of the ASD on in-service highways. The estimation is also carried out in two dimensions, both in plan and profile. It is usually examined on the stations defined by vehicle trajectory and vertical profile. According to the Spanish guidelines for highway geometric inventories (Ministerio de Fomento, 2009), stations are spaced 10 m apart. On the horizontal projection, obstructions are identified on orthophotos and the corresponding line-of-sight beam is launched to evaluate the ASD. On the vertical projection, the roadway profile itself determines the ASD at each station using current values of driver's eye height and target height as inputs. Then, the minimum of both values is taken as the definitive ASD at each station.

2.2.3.1.2. *Three-dimensional estimation methods*

The use of analytical formulae that relate the ASD to parameters featuring elements of horizontal alignment is possible in few individual cases only. For this reason, alternative methods have been conceived in 3D, mainly consisting of on-site measurements, real photographs or computational modelling.

Several examples of on-site measurements can be found in literature. In a study about sight orientation (Lippold and Schulz, 2007), the ASD on rural highways was determined with two vehicles equipped with GPS systems. Sight distance was checked at stations spaced 50 m apart and, in low visibility areas (curves or crests), at intervals of 25 m. To estimate the ASD on motorways, Steinauer et al. (2002) devised a measurement method based on the images from the driver's perspective. The ASD was posited to be the distance between the camera and the furthest visible lane marking. With a corresponding focal length set and clear view, the authors state that the guidelines are sufficiently recognizable. The superposition of all differences led to an overestimation of actual ASD in the range 8 - 18%. According to the authors, the reasons for this deviation of the measured values from the actual ones are the error of the observer point location (vertical and horizontal) and the target point location in comparison to the real trajectory.

In most cases, a procedure that computes of line-of-sight loops results more practical. Such routines consist of launching lines of sight from the driver's eye theoretical position to the target object, checking whether or not the line of sight is actually obstructed by the surface modelling the highway and its roadsides. The parameters and considerations made in 2D, such as the observer's height, the target height or their location in the cross section, are also necessary in 3D computation of ASD. Alternatively, the target point has been replaced by a target area (Charbonnier et al., 2010).

Hassan et al. (1996a) devised a 3D analytical model to calculate the ASD based on finite elements which represent the theoretical surface of the road, as well as cut side slopes, roadside obstacles, overpasses and other roadside features. In addition, the procedure permitted to confirm that 2D analysis may overestimate or underestimate the ASD. In further developments, the methodology was added more features (Hassan et al., 1998a).

Charbonnier et al. (2010) developed software QT-ballad, including a procedure for the estimation of the ASD on highways. This procedure comprised the highway survey with LiDAR equipment mounted on a car, besides other devices to collect 3D clouds of points massively from the highway and its surroundings. The surface model was applied an eventual decimation step to reduce significantly the computation time. Within the same project, a procedure for the estimation of the ASD using stereovision on paired images was developed. It was based on a photogrammetric restitution of the roadway contour to draw up a 3D model of the roadway surface. An automated algorithm adjusted the profile and facilitated the extraction of vertical alignment parameters. Finally, the ASD was deduced as the maximum focus distance for which left and right borders of the roadway surface can be adjusted through the restitution process. The main drawback of such a technique is the high sensitivity of results to existing traffic and weather conditions while sampling, which may underestimate the actual ASD. To compare those two measures of ASD, a third method using two cars checked the actual available sight distance, similarly to Lippold and Schulz (2007). It confirmed the better accuracy of the methodology based on LiDAR.

Castro et al. (2011) designed a process on a GIS to estimate the ASD using viewsheds. To put this procedure into practice, it is necessary to have a digital elevation model (DEM). In the study described, a raster DEM was utilized. Additionally, it requires the theoretical vehicle path, which

could be obtained either from alignment data in the construction project or from adjusted coordinates (x, y, z) derived from real trajectories travelled by a vehicle equipped with a GPS device onboard (Castro et al., 2006b). Also based on a GIS and using DEM's, Bassani et al. (2015) proposed a methodology to evaluate the ASD on urban arterials. Jha et al. (2011) addressed the problem of estimating the ASD also using viewsheds on MATLAB. Afterwards, Castro et al. (2014a) developed GIS-based software that is able to handle triangular irregular networks (TIN) datasets as DEM. From this information aggregated within a GIS platform, it is not only possible for researchers to study sight distance, but also to analyze other aspects of geometric design of highways, traffic operation and safety (Castro et al., 2008a; Castro et al., 2008b; Leroux, 2004; Saura, 2010). Another stated advantage of the techniques implemented on a GIS is that they permit the analysis of in-service highways even if there is no available or reliable data, as occurs, for example, when the alignment in project is amended during the construction phase, or when the highway alignment has been upgraded (Castro et al., 2011; Castro et al., 2014a).

Several authors have studied the effect of features of elevation models and other inputs on the sight distance output. For example, Castro et al. (2015) studied the influence of spacing between successive stations along the vehicle path on the results of sight distance estimation as well as the effect of the resolution of DTM combined with different spacing distances between stations. They warn about the loss in accuracy as the spacing between stations rises and particularly as the DTM resolution increases. The use of DSM's has advantages over DTM's as the first ones are able to take vegetation or near buildings into consideration (Khattak and Shamayleh, 2005). Gavran et al. (2016) devised software that provides accurate 3D ASD computation on highways, which is able to include modelled landscape items and featured sight distance restrictions. Moreover, Llobera (2007) advocates the consideration of vegetation in the evaluation of sight distance.

Campoy-Ungria et al. (2012) developed a procedure to compute ASD through clouds of points derived from car-mounted LiDAR surveying. Sight distance is checked by means of a pair of vertical planes enclosing the lines of sight. The offset between such planes, called "a", is a factor to be chosen and the space between them is divided into prismatic cells with dimensions "a" x "b" x "b". Hence, those cells which contain points inside are considered to be obstructing that line of sight. Unlike the previous methods, this one does need to rely on surfaces to check sight distance. Nonetheless, lines of sight may slip through points if the point cloud density is low even though the prismatic cell dimensions have been well studied. In other words, the method may overestimate ASD, lacking robustness against data features.

On board LiDAR technology has two substantive advantages. First, it enables a massive surveying with the highest performance compared to classical topography (Glennie, 2009). In addition, LiDAR remote sensing systems deliver measurements with relative error as low as a few centimeters, both on planimetry and altimetry (Barber et al., 2008).

2.2.4. Inverse sight distance

As explained in previous sections, the computation of sight distance requires that lines of sight are defined. Each line of sight is associated to two ends, namely the observer and the target. Commonly the ASD corresponds to the first end, the observer. However the consideration of ASD from the target location supplies additional and information, especially relevant for

highway safety research. In the case where sight distance is evaluated on the centerline, as suggested for example by the AASHTO guide (2011), the direct ASD corresponds to the inverse ASD on the opposing travelling direction at every location.

Collisions striking an object on the roadway or off-road excursions are likely to end up with the involved vehicle closer to the target location. Moreover, authorities assign the accident location at the final position of the involved vehicles in accident reports. In this respect, it is worth mentioning that Dadashova et al. (2016) incorporated direct and inverse ASD in prediction models to identify accident patterns on highways.

2.3. Relation between sight distance and driving behavior

2.3.1. Driving behavior

Highways are designed to be understandable by drivers, so that correct decisions are performed by them while travelling along a section. In this context, the driver workload is defined as the amount of information that a driver has to process per unit of time. Easa and He (2006) concluded that the driver's visual demand increases with curvature on both projections. These concepts have much bearing on safety. If the demanded workload exceeds the driver's capacity, an undesirable event may occur.

In addition, numerous studies about the relation between driving behavior and factors such as speed, sight distance, 3D alignment coordination or design consistency are available in literature. For example, Hasan et al. (2005) identified ASD as the most important factor affecting the perception of the alignment by drivers.

Diverse authors researched the influence of speed on driver's visual behavior on tangents (Babkow, 1975; Serafin, 1993). There, the driver tends to stare at an increasingly smaller area ahead near the vanishing point of the road in perspective as the speed increases (Figure 2.4). Several publications linked the driver's speed and their distance of gaze in the range of values from 30 to 100 km/h (FGSV, 1995a; Hamilton and Thurstone, 1937) and at higher speeds (Kawczyński, 1994).

Figure 2.4. Stare areas at different speeds (Babkow, 1975).

Concerning driving behavior and speed, Leutner (1974) defined three zones of different importance for the visual perception:

- Zone 1: long-distance orientation and information (between 250 and 600 m ahead). The definition is based on the ability of the human eye to recognize moving objects on the roadway at a distance of up to 600 - 800 m.
- Zone 2: Readiness and decision zone (between 75 and 250 m). The driver saves the information from the road characteristics ahead for a lapse of around 10 s. The driver may need to recognize and assess the roadway section ahead along which they will drive during the next 10 s. A distance of approximately 250 m is assumed to be travelled during such a period of time.
- Zone 3: close orientation and operating zone (less than 75 m). Drivers' start to reduce their speed about 3 seconds before reaching a stitching point on the alignment.

Several studies have been performed to provide some insight into the influence of sight distance on driver's speed. It was primarily found a weak influence of sight distance on driving speed (Lippold, 1997). In a further study, it was found that, in straight sections with clear roadsides, drivers tend to slow down significantly if the vertical alignment limits the ASD below 200 m. On the contrary, curve radius is much more decisive for speed selection than the ASD on horizontal curves (Lippold and Schulz, 2007).

2.3.2. Visual perception capabilities

Visual perception has a remarkable accuracy within a cone angle of 50° or 60°, it being satisfactorily well until 180°. For a range greater than 100°, visual acuity decreases significantly. A cone of vision with a vertical opening angle of 20° and 30° horizontally at its vertex is known as the useful field of view and the most accurate vision is achieved within 2 to 4° (AASHTO, 2010).

The maximum range of human vision is particularly important to analyze the length of headway that affects driver behavior at a given moment. The range of human vision in driving is a matter where many concurrent factors are present. The most relevant ones are the visual acuity, followed by the chromatic features of the target, the lighting up quality and the contrast of the environmental (Ketvirtis and Cooper, 1977). As it was addressed in Section 2.2.3.1, guides consider that the ordinary visual acuity is 1 sexagesimal minute arc in design. In addition, the current Swedish design standard (Vägverket, 1994) state that under bright light conditions, 1 sexagesimal min arc is the minimum angle needed to allow a driver with 20/20 static visual acuity to perceive the target object. This value is, as a matter of fact, the most widely accepted in many different backgrounds (Casas, 1994). According to such a criterion, a 0.2 m-high target object would be visible from a distance of 688 m and a 1.1 m-high vehicle from 4,125 m. However, the effect of the abovementioned factors is decisive, decreasing sharply such perception distances. In addition, there is a great deal of controversy concerning the use of these values (Fambro et al., 1997; Fitzpatrick et al., 1998; Harwood et al., 1995). Indeed, diverse studies aimed at retrieving the values of such distances. Ketvirtis and Cooper (1977) studied the drivers' performance when encountering diverse obstacles on the roadway in closed-loop facilities and under different lighting conditions, confirming that the driver misjudges collision risk. Janoff (1990) studied sight distance at nighttime on different illuminated streets. Diverse 0.2 m-high

objects with varied contrast features with respect to the pavement were used. Both authors obtained quite different detection distances depending on the viewed object and, in any case, much below the abovementioned 688 m.

The resolution capabilities of the human eye are finer for moving objects than for static ones. With respect to oncoming traffic, the favorable effect of relative movement makes vehicles recognizable from longer distances, a fact that does not happen among vehicles moving in the same direction. Hristov (2009) asserted that the upper limit for eye fixation from a psychological point of view was determined to be 500 m. Leutner (1974) estimated that the maximum distances from which moving obstacles can be noticed range from 600 to 800 m, and may be reduced to 400 or 500 m depending of the object size. Hiersche (1968) concluded that the maximum sight distance for traffic-related objects ranges from 700 to 800 m.

2.4. Three-dimensional alignment coordination

2.4.1. General principles

Ideally, highways should present a pleasing appearance, fit adequately into the environment and integrate with the landscape as viewed from outside the highway. Designers must bear in mind that the driver's perspective of the highway differs substantially from design drawings. Elements on the horizontal and the vertical projection should be adequately combined so that, in conjunction with the cross section, they give rise to a 3D whole free of undesired visual flaws.

The importance of achieving a good three-dimensional coordination on highway alignment is acknowledged by designers. Many principles of spatial alignment coordination are based on experience and agreement among professionals and definitely not on safety performance as it is difficult to establish statistically significant relations between accidents and the presence of such design deficits. In this section, the international experience on ASD-based analysis methods, as well as the study of 3D alignment coordination of highways and the possible interaction with safety is reviewed.

Sometimes, in the preliminary geometric design, the adjustment to the topography is more thorough in profile if compared to that on the plan or vice versa. An unfavorable superimposition of horizontal and vertical alignments may yield visual discontinuities or insufficient recognizability of the alignment, even though conditions imposed to horizontal and vertical alignment are separately satisfied (Lamm et al., 1999). These issues gain significance when it comes to two-lane highways.

Diverse geometric design guides and standards (AASHTO, 2011; Austroads, 2003; FGSV, 2012; Ministerio de Fomento, 2016; VSS, 1991) provide recommendations to arrange the highway alignments in plan and profile according to certain general rules. The aim is to preclude the existence of visual shortcomings in the highway final design. Although each document focuses on diverse nuances, one general ground rule is shared by most guides: the superimposition of segments in the vertical and the horizontal projection. Examples include the Spanish guide (Ministerio de Fomento, 2016). It encourages the overlap of profile stitching points on transition curves, located as far from the point with infinity radius as possible. Both the AASHTO guide (2011) and the Swiss standard (VSS, 1991) agree to advocate the coincidence of vertices in

horizontal and vertical projections (Figure 2.5.a). This way, an aesthetically pleasant three-dimensional s-shaped visual effect is created. Since this is not always possible, AASHTO (2011) admits the alignments to skip one phase as long as the vertex coincidence continues forward (Figure 2.5.b).

Figure 2.5. a) The standard case of coordination between horizontal and vertical alignment; and b) a legitimate case of coordination where one phase is skipped (AASHTO, 2011).

Related to the AASHTO criterion, Moreno et al. (2014) demonstrated that minimum ASD can be maximized where horizontal turns and crests are superimposed if the displacement between vertices is within a certain range.

In this context, Weise et al. (2002) developed the concept of standard spatial elements (SSE) to broach the 3D alignment coordination. They sought to provide a good recognizability of horizontal curves. By definition, a highway section is composed of a sequence of SSE if the start and end of horizontal curves coincide with the start and end of crests and sags, considering also horizontal tangents and grades as null-curvature turns. When checking such a rule, an offset between the starts and ends of horizontal and vertical segments is permitted (Figure 2.6). Spirals are effectively considered along with the adjacent circular curve as a single element. Weise et al. (2002) proposed a maximum admissible offset of 20% of the length of the horizontal segment regardless of the segment types on either direction. Notwithstanding the foregoing, they recommended that crests do not exceed the bounds of the horizontal curve they superimpose, and sags contain within their bounds the horizontal curves when superimposed. The criterion of the 20% offset was first set in the guidelines for visualization of rural highways, H ViSt (FGSV, 2008a). It was revised and updated in new guide of highway geometric design, RAL (FGSV, 2012), including any tangent-curve sequence superimposed on a single grade within the SSE definition, since such a sequence is supposed not to yield any perspective shortcoming.

A second basic concern, also shared by many design guides, is the need for a convenient relation between curvature on the horizontal and the vertical projection. For example, AASHTO (2011) recommends that ample radius values in plan are accompanied by comparable curvature values on profile, without mentioning what is meant by comparable. Conversely, the Spanish instruction (Ministerio de Fomento, 2016) offers a more practical rule. It establishes the value of vertical curvature whenever design is equal or below 60 km/h as the ratio radius – superelevation if possible, or set the ratio $K_v - R$ over 6 otherwise. Nonetheless, Moreno et al. (2014) utilized the ASD as a criterion to optimize this particular case of 3D alignment coordination. It was observed that required SSD was satisfied when the ratio $K_v - R$ is below the abovementioned standard in Spain.

Figure 2.6. Sequence of standard spatial elements (FGSV, 2008a).

Some guides provide additional rules that pursue the enhancement of visual quality and legibility of the roadway. For hilly terrains, the former German guide (FGSV, 1995b) recommends that the crest parameter is greater than the one of the sag curvature where adjacent. This way longer ASD is provided to drivers. On the contrary, for rather flat terrains (gradients around or below 10 m), it is advisable that sags are smoother than crests (Figure 2.7). Thus, the driver will enjoy a more satisfactory view of the immediate section ahead (Lamm et al., 1999).

Figure 2.7. Sequences of alignments on the vertical projection (FGSV, 1995b).

2.4.2. Design flaws in the alignment

The lack of proper spatial coordination among alignments on the horizontal projection and those on the vertical projection may lead to visual distortions that could mislead or violate driver's expectations (Kraemer et al., 2009).

Different cases of visual distortion are described throughout guides worldwide. The cases listed below are commonly classified into two groups. The first group comprises safety-related flaws, which are thought to jeopardize safety (Kühn, 2013). The second group corresponds to minor shortcomings, mere aesthetical flaws that might hardly pose hazard for drivers.

2.4.2.1. Safety-related flaws

2.4.2.1.1. Hidden beginning of horizontal curves

When the beginning of a horizontal curve is located in the vicinity of the top of a crest, the driver can hardly perceive the horizontal change in alignment sufficiently in advance (Zimmermann, 2001). The difficulty in recognizing this spatial sequence of alignments is especially aggravated during nighttime (Hassan and Easa, 1998). Drivers should be able to perceive the existence of the curve and decelerate at a comfortable rate before entering the curve. A substantial part of the curve should be seen so that the driver can perceive the horizontal deflection. The French design guide (SETRA, 1994) suggests a horizontal turn should be perceived at least at a distance travelled during 3 seconds at the speed v_{85} , yet it does not define how much of the turn must be seen. Conversely, the German guides (FGSV, 2008a; FGSV, 2012) consider that not only the beginning of the turn has to be perceptible, but also a stretch where the heading variation exceeds 3.5 gon.

The Spanish standard (Ministerio de Fomento, 2016) determines that the stitching points of crests and sags, where overlapped with a horizontal curve, must lie within the corresponding spiral, and as far from its infinity-radius point as possible.

Based on previously developed studies (Zimmermann, 2001; Zimmermann et al., 2007), a more precise criterion to quantify the offset than that of the SSE is also proposed by the H ViSt (FGSV, 2008a). The most unfavorable case would comprise a grade on a tangent followed by a horizontal curve superimposed on a crest vertical curve. It is required that, if the tangent overlaps the crest, the horizontal curve starts close enough to the crest bound to make the horizontal bend perceptible for the driver. A curve is considered perceptible when the significant change in heading angle of 3.5 gon ahead is foreseeable from a distance equal or larger than 75 m (FGSV, 2008a). The driver must find the end of the tangent closer than the beginning of the crest, at a distance at least equal or greater than that exhibited in Table 2.2 as a function of the crest parameter K_V and the spiral parameter A .

Similarly, AASHTO (2011) discourages the existence of sharp horizontal curves at or near the top of a crest vertical curve. In this regard, Hassan and Easa (1998) discussed the coordination of crests and horizontal curves quantitatively. They defined the locations where a horizontal turn should not begin in relation to a vertical crest as sight-distance red zones because certain superimposition offsets can lead to poor perception conditions. Daytime and nighttime circumstances were considered in that study whereas horizontal radius, vertical curvature, change in direction, SSD and preview sight distance (Gattis and Duncan, 1995) were included as discussion variables.

Table 2.2. Minimum offset required for horizontal curves on crests (FGSV, 2008a).

Vertical curve parameter, K_V [m]	Spiral parameter, A [m]			
	150	200	250	300
-3000	25	50	65	80
-4000	15	35	55	75
-5000	No limited offset required	25	50	70
-6000		15	40	60
-7000			30	55
-8000			20	45
-9000			10	40
-10000				30

When curve perception is not fair enough, chevron signs can be placed to warn about the existence of the concealed curve. They can be seen from a distance much further than the pavement markings.

Figure 2.8. Shift from tangent to curve in plan hidden by a crest (VSS, 1991).

Figure 2.9. Shift from left turn to right turn in plan hidden by a crest (Ministerio de Fomento, 2000).

2.4.2.1.2. Sight-hidden dips

Sight-hidden dips are a type of perspective flaw caused by certain combinations of alignments in plan and profile. They may be found where a crest vertical curve is followed by a sag vertical curve on a rather straight horizontal alignment. The field of vision is partially limited by the topography on crest vertical curves. At these spots, the driver is able to see the roadway ahead

and simultaneously another farthest section but not the one in the middle. They indicate a too generous design in plan against a more strict profile (Kraemer et al., 2009).

Sections where sight-hidden dips exist are locations where the ASD is often below that required for emergency stopping or passing manoeuvres. Drivers may not be aware indeed of the uncertainty beyond the ASD. Hence these sections carry certain risk for overconfident drivers if they infer that they can see any oncoming vehicle when checking the opposing lane to initiate a passing maneuver, yet the hidden section could conceal an unnoticed vehicle.

First, it is worth to mention that Austroads (2003) recommends to dispose as few tangents as possible, connected through smooth vertical curves, to prevent sight-hidden dips. These are merely descriptive guidelines that can hardly aid designers.

Conventionally, sight-hidden dips have been characterized in guides through perspective depictions. The German standards report sight-hidden dips comprehensively as examples of flawed design (FGSV, 1995b; FGSV, 2012). Sight-hidden dips are classified into several groups according to the existing horizontal alignment: *diving* if the horizontal alignment is essentially straight (Figure 2.10.a) or if it has one change in direction within the hidden area (Figure 2.10.b), *jumping* if there are changes in direction towards each side (Figure 2.11.a) or *fluttering* if there are several hidden areas in a row perceived simultaneously (Figure 2.11.b).

Figure 2.10. Sight-hidden dips a) on a tangent y b) on a horizontal curve (FGSV, 1995b).

Figure 2.11. Sight-hidden dips: a) horizontal sway or jump, and b) flutter (FGSV, 1995b).

Unlike the previous rules, a more quantifiable approach is set by the Swiss norm (VSS, 1991). It establishes that sight-hidden dips are to be avoided in design where intersections and relevant changes in direction are found within the hidden area. In order to reduce the hazard of dips otherwise, this standard defines a minimum operating distance which depends upon the design speed. It is displayed by the solid line in Figure 2.12. Thus, the reemerged roadway beyond the blind spot must lie at a distance greater than this threshold (area "1" in Figure 2.12). The Italian

standard (Ministero delle Infrastrutture, 2001) provides a practically identical list of values for the same purpose.

Figure 2.12. Required distance to the end of the hidden section (VSS, 1991).

The line defined by the graph in Figure 2.12 was probably devised according to operational criteria based on PSD. However, more criteria adjust to the threshold thereby defined, which are shown in Figure 2.13. Indeed, as detailed in section 2.3.1, several publications link the driver's stare distance and the speed. (FGSV 1995a; Hamilton and Thurstone, 1937; Kawczyński, 1994).

The different criteria pointed out can be compared in Figure 2.13. It gathers the information above referred to and that of the Figure 2.12, as well as PSD of different standards. It exhibits how similar are the trends followed by the lines depicting the driver's gaze distances (dashed red, yellow and green lines), slightly above the solid black line defined by the Swiss standard (VSS, 1991). The values proposed by Hamilton and Thurstone (1937) are closer for speeds below 100 km/h. Finally, the graph shows how PSD criteria of four standards also follow the trend of the solid black line: former Spanish (Ministerio de Fomento, 2000), Italian (Ministero delle Infrastrutture, 2001), Swiss (VSS, 1991) and British (Department for Transport, 1993).

2.4.2.2. Aesthetical flaws

2.4.2.2.1. Visual distortions on the edges

When a left bend follows a tangent right beyond a sag vertical curve, an undesirable optical effect can be produced on the outer travelled way border, giving rise to a false appearance of a preceding right bend (Figure 2.14). Such a perspective distortion, besides unsightly, may result misleading for drivers if both the grade variation of the sag is great and the vertical curve is relatively short (Kraemer et al., 2009). Even the superelevation may aggravate the visual distortion.

Figure 2.13. Comparison among visual fixation and PSD.

Figure 2.14. Perspective of a sag curve preceding a horizontal bend (Kraemer et al., 2009).

Some geometric design standards (FGSV, 1995b; Ministerio de Fomento, 2016) include recommendations to smooth such a visual distortion by limiting the minimum threshold parameter of the vertical sag as a function of the horizontal radius. However, these recommendations may seem ambiguous when the horizontal and vertical bends are not overlapped.

2.4.2.2.2. Distorted perception of the curve radius

Where a sag curve and a horizontal bend are overlapped, the driver may judge that the curve radius is greater than it actually is (Smith and Lamm, 1994; Zimmermann, 2001). This distorted perception of the curve can make the driver to take the curve at an inappropriate speed. It was studied by Hassan and Easa (2003) using computer animation to simulate 3D alignments. The different behavior of drivers with different alignment profile was confirmed.

Figure 2.15. Perspective of a horizontal curve on a sag (Zimmermann, 2001).

2.4.2.2.3. False roadway narrowings

Another example of poor spatial alignment coordination is produced where the inflection point, i.e. the stitching point of reverse curves, in plan is located within a sag curve (Ministerio de Fomento, 2016; VSS 1991). Such an alignment arrangement yields a false narrowing appearance in the roadway perspective (Figure 2.16). It is therefore a rather aesthetic visual distortion that may mislead drivers.

Figure 2.16. Apparent roadway narrowing, a) horizontal and vertical projections and b) in perspective (VSS, 1991).

2.4.2.2.4. Sharp-bend spots

Those bends between two straight segments where a noteworthy deflection angle is turned compared to the curvature radius could create an appearance of an abrupt bend if perceived from a far enough distance. This applies to both curves in plan between two tangents and curves in profile between two grades. Moreover, the optical effect can be worsened if bends in both projections coincide on the same spot. In order to prevent these shortcomings in the perspective appearance, it is convenient that both curve ends fall outside the relaxed vision field (see section

2.3) from wherever the sharp bend is observable (Kraemer et al., 2009). Concerning the safety implications of horizontal sharp bends, the principles of design consistency point out such spots as hazardous since high speeds are reached on long tangents and therefore the speed reduction when approaching the curve must be pronounced (Lamm et al., 1999). Vertical sharp bends limit nighttime sight distance and passengers may experience uncomfortable vertical forces when travelling at high speeds.

Figure 2.17. Sharp horizontal bend (VSS, 1991).

Figure 2.18. Sharp bend in the longitudinal profile (VSS, 1991).

As a matter of fact, spirals are assumed to enhance the perception of horizontal curves (Austroads, 2003). The Spanish standard of geometric design (Ministerio de Fomento, 2016) includes recommendations to prevent the manifestation of sharp-bend spots in the alignment. In the horizontal projection, the insertion of spirals is compulsory for radii below 5,000 m (groups 1 and 2) and below 2,500 m (group 3). In addition, the Spanish guide limits to 700 m the minimum radius of a curve followed by a tangent longer than 400 m whenever the design speed is set at 100 km/h or higher. The restrictions on sag curves are based, besides nighttime ASD criteria, on aesthetical considerations as the length of the sag (in meters) is required to exceed in any case the design speed (in km/h).

2.4.2.2.5. Verticalization of grades

The Verticalization of grades is a particular case of sharp-bend spots. This shortcoming in the spatial alignment consists in the appraisal of the grade that follows a sag as almost vertical from the driver's perspective (Figure 2.19). To avoid this undesirable optical effect at or near sags, AASHTO (1994) recommended that the sag segment cover a length such that when seen at first it is at minimum 0.6 times the length of the prior element of the profile. Moreover, it was required that the minimum length (in meters) exceed three times the design speed (in km/h) and it was recommended that it span six times that length given by the design speed.

Related to the verticalization of grades, the smoothening visual distortion created by horizontal curves exceeding the ends of an overlapped sag may be considered. This phenomenon has been studied by several authors (Hassan and Easa, 2003; Smith and Lamm, 1994). In all its forms, this

shortcoming may cause that drivers increase their speed since they do not perceive the alignment adequately (Zimmermann, 2001).

Figure 2.19. Representation of a grade on a tangent (Kraemer et al., 2009).

2.4.2.2.6. Other visual shortcomings on the 3D alignment

The insertion of short alignments leads to visual flaws such as the so-called broken back effect. To avoid it, many standards (Austroads, 2003; FGSV, 2012) urge not to introduce short tangents nor short grades, but full-covering vertical curves instead that prevent the perspective abovementioned deficit. Similar recommendations are found in North America and Switzerland (AASHTO, 2011; Berger and Cron, 1977; VSS, 1991), where, besides that, vertical curves are encouraged to cover a length, in meters, six times longer than the design speed, in km/h.

AASHTO (2011) also discourages inserting small-radius curves in the vicinity of a crest curve, given the possible sight-distance related issues that may arise, especially at nighttime. It is likewise advisable not to create two visual breaks in plan nor three of them in profile at any spot.

2.4.3. Evaluation of the 3D alignment coordination

2.4.3.1. Computer-aided visualization techniques

The use of central perspective plots permit designers to go straight from the projected drawings to the virtual recreation of a highway. Indeed, perspective figures were used in the previous section to feature the 3D design flaws.

Computer-aided visualization techniques for highway perspective depiction first begun to develop in the sixties (Park et al., 1968). Although at first merely grid plots were available, the computational capabilities gave rise to enhanced representations of the projects. Today, commercial software for highway design comprises powerful 3D visualization modules to recreate the driver's perspective along a section (Castro, 2012; FGSV, 2008a; Kühn, 2013; Puy, 2014). This tools are increasingly more accurate and lifelike. Figure 2.20 exhibits one of such highway perspective depictions. The succession of perspective views forms an animation where the spatial design issues can be spotted. Furthermore, a driving simulator has been proposed as an additional implementation to check the quality of geometric design (Kühn et al., 2011). Nonetheless, these evaluation tools are at the expense of the supervisor's ability to judge, requiring an experimented person (Larocca et al., 2011; Osterloh, 1983). This way the spatial shortcomings might be readily observed and rectified.

Figure 2.20. Recreation of the highway perspective from the viewpoint of a driver by VESTRA software (Bark and Schmidt, 2010).

The use of visualization techniques has, presumably, a qualitative scope in the assessment of highway design. However, the principles of central perspective yielded diverse propositions from a quantitative point of view. They will be reviewed in the next section.

2.4.3.2. Quantitative methodology to study 3D alignment coordination

As it was explained in previous sections, design standards generally include guidelines to achieve proper 3D alignment coordination to aid designers. These recommendations are mostly given in qualitative terms. However, in recent years, research focused on rather quantitative aspects of this matter. Several authors have made an attempt to quantify the quality of geometric design through different approaches, yet their use has not been broadly accepted in standards.

As each shortcoming type presents a different level of hazard, the features of certain types of spatial design shortcomings can be quantified more easily than others.

Sight distance graphs, which are created from line-of-sight loops, may also be employed to study 3D alignment coordination. These charts represent the stations (locations where the driver is sequentially placed) in the abscissa, and the sight distance variables ahead each driver position as the ordinates. The features are either way measured along the theoretical vehicle path. This plot can be very advantageous when sight distance issues and sight-hidden dips have to be studied (Castro et al., 2014a; Zimmermann, 2001).

2.4.3.2.1. Characterization of sight-hidden dips

An initial approach to characterize sight-hidden dips was performed by Easa (1994a). He developed an analytical method assuming a relatively straight horizontal alignment to determine the relations among the geometric features of these profiles. Based on the vertical alignment parameters, the maximum depth q^* of the hidden section is given by:

$$q^* = \frac{K_{V,1} + K_{V,2}}{2 \cdot K_{V,1}^2} \cdot u^2 + \frac{d \cdot u}{K_{V,1}} \quad (2.17)$$

where:

$K_{V,1}$ = curvature parameter of the crest, m.

$K_{V,2}$ = curvature parameter of the sag, m.

u = distance from the tangency point between the sight line and the crest to the crest end (Figure 2.21), m.

d = length of the grade between the crest and the sag (Figure 2.21), m.

On spots where this maximum depth is shallower than the target height h_2 , the hidden area may be dismissed. The position of the driver T_s in relation to the beginning of the crest when they first detect the design shortcoming can be determined. Two cases are possible: the driver is able to see the end of the grade g_2 beyond the sag curve from the beginning of the initial grade g_1 ; and the driver cannot see that end from the beginning of g_1 (Figure 2.21). Similarly, the position of the driver T_e where the hidden dip is no longer perceived can also be deduced, and that point may lie either on the crest or on the grade g_1 prior to the crest. The following equations can be employed to evaluate both T_s and T_e .

$$T = \frac{z}{2} - \frac{h_1 \cdot K_{V,1}}{z} \quad \text{where } T \leq 0 \quad (2.18)$$

$$T = z - \sqrt{2 \cdot h_1 \cdot K_{V,1}} \quad \text{where } T > 0 \quad (2.19)$$

Besides the variables of Equation (2.17), the following ones are involved:

T = distance from the driver's position to the beginning of the crest ($T > 0$ if the driver is on the previous grade, Figure 2.21), m.

h_1 = driver's eye height over the roadway, m.

Figure 2.21. Geometric layout on a section where a sight-hidden dip exists (Easa, 1994a).

The analytical approach by Easa (1994a) provided worthy insight on the geometric relations among the geometric parameters of sight-hidden dips. However, it did not cover all possible alignment configurations. First, the author assumed that an unlimited grade always precedes the crest curve. Second, to obtain the position of the driver when they detect the visual break

in the roadway an arbitrary criterion is used: either furthest end of grade g_2 or the virtual intersection of grades g_1 and g_2 . From that assumptions, the range is deduced as the difference between the driver's initial and final position, substituting in Equation (2.18) or (2.19).

$$R = T_e - T_s \quad (2.20)$$

According to Easa (1994a), the key alignment parameters in the analytical study of sight-hidden dips are the lengths of the vertical curves involved (L_1 and L_2 in Figure 2.21) once fixed their respective curvature values $K_{V,1}$ and $K_{V,2}$ and the length of the grade between the vertical curves d . Zimmerman (2001) also studied, under an analytical approach, the depth of dips and the distance from the driver to the reemerged roadway on tangents, yet he assumed a different non-generic profile layout with. According to the author criteria, the depth of diving is critical where it exceeds 0.5 m since the reflective band of delineator posts is visible only at shallower dips.

According to the parameters defined by Easa (1994a) shown in Figure 2.21, the length of hidden section L_{HS} is determined by the following sum:

$$L_{HS} = u + d + L_1 + N_V \quad (2.21)$$

The length of the hidden section can alternatively be estimated on site by measuring featured distances on site georeferenced photographs on in-service highways (Castro et al., 2014b).

Finally, the distance from the driver to the end of the hidden section is determined by adding the length of hidden section (L_{HS}) to the ASD.

Easa (1994a) also reported that, where the driver is located on the initial grade g_1 , at a distance to the beginning of the crest (point B in Figure 2.20) greater than the pertinent PSD, the dip is not critical for safety. Finally, the author includes an evaluation of the AASHTO guide (1994) current in those days.

Sánchez Ordóñez (2011) proposed a numerical index to predict if a crest followed by a sag may create a sight-hidden dip, either or not overlapped with a tangent in plan. It is computed on each driving direction and its mathematical expression is given by:

$$I_{rt} = \frac{K_V}{(G_e - G_s) \cdot d} \quad (2.22)$$

where:

- I_{rt} = sight-hidden dip index, dimensionless.
- G_e = slope of the preceding grade, dimensionless.
- G_s = slope of the interim grade, dimensionless.
- d = length of the interim grade (Figure 2.21), m.

The index value is always positive on crests. For values lower than 25, it was concluded that the likelihood that a sight-hidden dip is produced was 41%, a value that is far from being even indicative. One of the possible causes of its low predictive effectiveness is that the formula

overlooks significant variables since the grade ahead the sag curve (g_2 in Figure 2.21) as well as the horizontal alignment are dismissed.

2.4.3.2.2. Quantitative analysis of highway perspective

Some authors proposed quantitative methods to evaluate the quality of the design based on measurements on the central perspective projection of the highway. First, Pfeil (1970) suggested the relation between the deflection angle of a curve and the observed turn in the perspective as an indicator of the 3D alignment coordination quality. Next, Leutner (1974) proposed to evaluate the goodness of spatial alignment coordination from the curvature radius of the perspective depiction of a curve, both on the centerline and on the travelled way edge. Springer and Huizinga (1975) suggested, moreover, another geometric method based on the depiction of the straight lines tangent to the hyperbola that plots the curve in central perspective. Finally, Osterloh (1983) studied mathematically the perspective representations of the highway. The advances in this area, besides arduous and cumbersome, result inefficient to scrutinize the spatial superposition of alignments (Zimmermann, 2001). Consequently, they will hardly be able to be implemented in highway spatial alignment checking.

Zimmermann (2001) devised a methodology to evaluate the 3D alignment also based on measurements of the highway perspective recreation. To this end, the proposed reference value was the ratio of visualized area of the actual roadway surface (Figure 2.22.a) to the visualized area of that same surface assuming flat profile instead (Figure 2.22.b). For example, a high value of such an indicator is expected at verticalization of grades on the grade that follows the sag. Moreover, this methodology enabled the evaluation of all the possible cases of sight-hidden dips presented in section 2.4.2.1.2, such as bulges, apparent smoothing in the perspective of curves overlapped with sags, and hidden beginning of curves.

Figure 2.22. Roadway perspective depiction a) as designed and b) with null grade.
(Zimmermann, 2001).

The outcome of the proposed analysis is displayed in the so called "QuaSi-Band" graph (Zimmermann, 2001). It shows the evolution of the visual outcome of the highway in perspective over a section enabling the evaluation of the alignment coordination. The results plotted include the ASD and the depth of diving in hidden dips.

To remove hidden dip sections or at least reduce the possible impact on safety, Zimmermann (2001) suggested either to reduce the depth of diving or to move away the reemerged area

further than 800 m. Both redesign measures can be put into effect by smoothing the grade between the crest and the sag and raising the sag with respect to the crest, keeping the vertical curve parameters constant.

2.4.3.2.3. Evaluation of hidden beginning of curves

Zimmermann (2001) proposed new geometric design guidelines to prevent shortcomings in the spatial alignment at the design stage. First, the maximum offset between the beginning of a crest and the beginning of an overlapped horizontal curve is recommended. Particularly, the criterion indicates that the initial part of the curve (3.5 gon) must be foreseen 75 m before the driver steers into the curve. This instruction aims at providing recognizability of the horizontal bend sufficiently in advance. A similar study was carried out by Hassan y Easa (1998) defined the so-called red zones on crest curves, meaning locations where horizontal curves must not be inserted.

2.4.3.2.4. Evaluation of distorted curve perception

One of the advantages of the methodology devised by Zimmermann (2001) using the QuaSi-band graph is the capability of detecting and analyzing horizontal curves that may seem smoother if they are overlapped with a sag. If the sag curve exceeds the ends of the horizontal curve, Zimmermann (2001) proposed the use of a cubic parabola in the case the visual distortion results severe. As the curvature of this parabola varies quicker, the vertex seems sharper in the perspective so the distortion would be reduced.

2.4.3.3. Methodology of the German standard

The German guidelines for the visual recreation of highways (H ViSt) (FGSV, 2008a) deserve particular attention in this respect. The principles and criteria given are based on research findings reviewed in previous sections (Leutner, 1974; Weise et al., 2002; Zimmermann, 2001). It also proposes a procedure to check the compliance of highway design with spatial alignment coordination criteria. These guidelines apply to all projects of brand new two-lane rural highways in Germany, regardless of the road category or the expected traffic load. Unlike other standards, German guidelines do not consider the concept of design speed in this respect.

The H ViSt guidelines develop an iterative procedure comprising three phases. It requires a series of checks to be performed which may demand changes in the alignment attributes until the standard is met. First of all, it must be possible to split the whole alignment into a sequence of SSE (Weise et al., 2002). When this condition is satisfied, the only check to be made is to confirm that no critical sight-hidden dip persists to the driver's perspective. A sight-hidden dip is considered critical where the following conditions concur.

- The range exceeds 60 m.
- The longest hidden section exceeds 75 m, according to Leutner (1974) (see 2.3).
- Depth of diving exceeds a 0.75 m (Figure 2.23).
- The reemerged stretch is closer than 600 m (Figure 2.23), also in line with the provisions of Leutner (1974).

If the alignment cannot be completely divisible into a sequence of SSE neither is it feasible to alter its layout, it must be verified that no hidden beginning of a curve is produced, namely curves where a 3.5-gon turn is not perceptible at least 75 m before entering the curve (see section 2.4.2.1.1). If any, the alignment must be redesigned. Then, the designer must check that no aesthetical flaw is produced, for example sharp bend spots, broken back effect or bulges. As alignment redesign in plan require additional land expropriation, recommendations foster enlarging vertical curves and reducing the grade slope while keeping balance of the volume of earth moved.

Figure 2.23. Critical hidden section according to the German standard for highway visualization (FGSV, 2008a).

A summary of the most relevant sight-hidden dip parameters found in literature are shown in Table 2.3.

2.4.3.4. Remedial actions at sight-hidden dips

Alignment upgrade to remove geometric design flaws is often not possible or feasible on in-service highways. Furthermore, Glennon (1987) stated that redesign of already-existing facilities is beneficial only on busy highways where important hazards have been acknowledged, such as intersections or sharp curves. Then, alternative countermeasures must be contemplated. They may range from warning drivers about these potentially dangerous sections to limit speed or passing maneuvers. According to the definition of ASD, passing should be prohibited if this distance does not meet the minimum required to guarantee such maneuvers are accomplished safely. For this reason, solid road markings and vertical traffic signs ought to forbid passing at the beginning of crests wherever ASD is not long enough, regardless of whether a sight-hidden dip exists or not.

Table 2.3. References to relevant sight-hidden dip parameters in literature.

Parameter	Authors that referred to	Critical values
Length of hidden section	Easa (1994a)	-
	German standards (FGSV, 2008a; FGSV, 2012)	> 75 m
	Zimmermann (2001)	-
Maximum depth of hidden area	Easa (1994a)	-
	German standards (FGSV, 2008a; FGSV, 2012)	> 0.75 m
	Zimmermann (2001)	> 0.50 m
Visible range	Easa (1994a)	-
	German standards (FGSV, 2008a; FGSV, 2012)	> 60 m
	Zimmermann (2001)	-
Distance to end of hidden section	German standards (FGSV, 2008a; FGSV, 2012)	< 600 m
	Swiss standard (VSS, 1991)	< 180 to 860 m
	Italian standard (M. Infrastrutture, 2001)	< 150 to 860 m
Minimum ASD	Easa (1994a)	< PSD
	Zimmermann (2001)	

2.4.3.4.1. Blind-summit and hidden-dip signposts

Unlike other standards or guides, Standards for British highways (Department for Transport, 1993) do not contemplate the occurrence of sight-hidden dips at the design stage, even less so provide guidelines in order to avoid such design flaws or reduce their hazard. However, a specific vertical traffic sign can be found to warn against the presence of this design shortcoming (Department for Transport, 2013). Nonetheless, no proper rules to set out this sign are provided, nor has research supporting the convenience of warning about the presence of this alignment shortcoming been found.

Figure 2.24. Signposted long sight-hidden dip where, conversely, passing is not prohibited (Dixon, 2011).

2.5. Operating speed

2.5.1. Speed concepts

Speed is another essential factor in highway operation and geometric design. A distinction must be drawn among design speed, posted speed and operating speed. Design speed is a value given for an entire section by the Administration at the project stage. The highway operation and the impact on the environment depend largely on it. Similarly, posted speed is a segment-wise fixed upper legal limit that affects a section or subsection. On the contrary, operating speed is the distribution of speed values chosen by the drivers, different on each point of the highway alignment. A close likeness among them all is nevertheless highly desirable (Fitzpatrick et al., 2003).

The operating speed range of values can be characterized in several ways. The 85-th percentile value is commonly employed for example in design consistency studies. It is defined as the speed below which 85% of vehicles operate under free flow conditions, on clean wet road surfaces. Nonetheless, the information this definition provides about the distribution of speeds is virtually nonexistent. When the speed is to be treated as a random variable, more information is necessary. This is the case of studies performed under a probabilistic approach (reviewed in section 2.7), where statistical facts that characterize the statistical distribution are needed.

2.5.2. Speed prediction models

Speed prediction models are mostly based on geometric design features. Whereas predicted speeds are a function of local characteristics, some studies acknowledge a certain influence of the general character of the alignment (Cardoso, 1995). Based on the mentioned percentile values, these models usually assume constant speed on certain alignments: speed on circular arcs and on long tangents. The relation between those speeds and geometric features is inferred based on field data through linear regression models. One of the most frequently used independent variable is the curvature change rate (CCR), which includes the influence of the adjacent transition curves. It is calculated according to equation (2.23):

$$CCR = \frac{\Omega}{L_T} = 31811 \cdot \frac{2 \cdot L_{CC} + L_{TC1} + L_{TC2}}{L_T \cdot R} \quad (2.23)$$

where:

CCR	=	curvature change rate, gon/km;
Ω	=	deflection angle, gon;
L_{CC}	=	length of circular curve, km;
L_{CT1}, L_{CT2}	=	length of transition curves, km;
R	=	radius of circular curve, m;
L_T	=	overall curve length (Equation (2.24)), km;

$$L_T = L_{CC} + L_{TC1} + L_{TC2} \quad (2.24)$$

The curve radius R is the second most used parameter to deduce the speed on curves. The degree of curvature is also commonly found in operating speed modelling. It is defined as the angle subtended (gon) per length unit on the curve. The length of the curve or the spiral parameters are often added as secondary independent variables to regression models. Besides linear regression, other techniques such as artificial neural networks have been utilized to predict speeds on curves, overcoming nonlinearities or interaction effects inherent to the use of linear regression (McFadden et al., 2001).

Separated regression models for different cross sections, considering commonly radius as independent variable, can be found in literature (Lippold, 1997; SETRA, 1994). In the case of the former German standard (FGSV, 1995b), the asymptotic speed when the radius tends to infinity is assumed to coincide on each model. Lamm and Choueiri (1987) utilized the CCR and the degree of curvature to predict operating speeds on curves in Europe and the US considering different lane widths.

Vertical alignment also deserves attention with regard to speed prediction models. Swiss standard (VSS, 1991) outlines the construction of two separate operating speed models for the horizontal and the vertical alignment, assuming eventually the minimum speed at each point when merging both profiles. Similarly, SETRA (1994) provides complementary operating speed principles on either alignment projection according to the radius and grade respectively. Some more complex models take account of vertical alignment features combined with those on the horizontal alignment (Fitzpatrick et al., 2000a).

Research studies attributed little influence to ASD on operating speed (Lippold, 1997). Krammes et al. (1995), although recommended including sight distance in the speed-profile model, did not consider that practical.

Incorporating heavy vehicles provides a more comprehensive understanding of operating speed models. Donnell et al. (2001) developed a speed prediction model for heavy vehicles, concluding that vertical alignment has a greater influence on heavy vehicle speed than horizontal alignment. Dey et al. (2006) studied bimodal speed distribution curves to characterize operating speeds. Additionally, standards provide formulation for heavy vehicle speed reduction on grades (FGSV, 2015; Ministerio de Fomento, 2016). They normally assume that the steeper the uphill grade is, the more pronounced the deceleration undergone is (Figure 2.25).

In order to better characterize drivers' speed choice on highways, operating speeds under unfavorable environmental conditions have also been subject of study. For instance, Rakha et al. (2008) quantified the impact of adverse meteorological conditions, namely precipitation and visibility, on free-flow speed. Lamm et al. (1990) compared operating speeds between dry and wet pavements of curved roadway sections of two-lane highways. Hong and Oguchi (2005) confirmed a significant difference between daytime and nighttime operating speeds in Japan.

As mentioned earlier in this section, to characterize the statistical distribution of speeds, a central tendency value and a measure of deviation provide more complete information of the operating speed. In this regard, Figure 2.26 shows a comparison of the 50-th percentile speed on curves proposed by Richl and Sayed (2006), Castro et al. (2006a) and Pérez-Zuriaga (2012) and the corresponding standard deviations. On curves with radius greater than 100 m, the

model of Castro et al. (2006a) predicts the highest speeds, yet the differences among regressions are rather low. It conversely reveals that the speed adjustment of the red and green lines, projected on the ordinates, are far different for radii below 100 m. In fact, the output speed values of Castro et al. (2006a) model, especially below 70 m, do not even appear to be sound nor realistic. With regard to the standard deviations, whereas the trend on the green and red dashed lines is to increase over radius, the blue dashed line decreases at the smallest radii and remains approximately constant when the radius is greater than 400 m.

Figure 2.25. Heavy-vehicle speed reduction/increase according to the Spanish standard (Ministerio de Fomento, 2016).

Figure 2.26. Comparison of 50-th percentile speed and standard deviation curves.

2.5.3. Creating operating speed profiles

Devising continuous speed profiles is advantageous in design assessment or road safety research. Pérez-Zuriaga et al. (2010) deduced operating speeds on curves as a function of the CCR, the radius and the spiral parameter through on-board GPS devices in Valencia (Spain). They obtained a better fit with the first variable. Castro et al. (2006a) conceived an operating speed model measuring speeds on two-lane rural highways located in Madrid (Spain).

McLean (1981) defined desired speed as the speed at which a driver wishes to travel a road section under free-flow conditions when they are not constrained by alignment features. Although desired speed is assumed to be reached on long enough tangents, less attention has been paid to this aspect in research. Research showed that the length of the tangent determines its 85-th percentile speed (Ottesen and Krammes, 2000). According to Cardoso (1995), desired speed on both curves and tangents depend on cross section features, namely lane width and shoulder width. The most important explanatory variable is the radius of the preceding curve though. Conversely, Fitzpatrick (2000a) stated that the roadway width did not affect drivers' speed choice neither on tangents nor on curves. McLean (1981) maintained that desired speed is related to the terrain type and especially on the approximate range of horizontal curve radii (Table 2.4).

Table 2.4. 85-th percentile values of the speed environment (McLean, 1981).

Range of curve radii [m]	Terrain type			
	Flat	Undulating	Hilly	Mountainous
$R < 75$			75	70
$75 < R < 300$		90	85	
$150 < R < 500$		100	95	
$R > 300 \sim 500$	115	110		
$R > 600 \sim 700$	120			

Transition between elements having different speeds is usually assumed to be performed at a constant acceleration and deceleration rates. The Swiss standard (VSS, 1991) recommends a uniform acceleration rate of 0.5 m/s^2 and a deceleration rate of -1.0 m/s^2 . In contrast, many authors (Castro et al., 2008b; Krammes et al., 1995; Lamm et al., 1988; Ottesen and Krammes, 2000) propose symmetric values of both at a rate of 0.85 m/s^2 . Nonetheless, Fitzpatrick et al. (2000a) modelled variable acceleration and deceleration rates. Whereas deceleration was found to be performed at a rate that decreases linearly with the inverse of the radius, the recommended acceleration model was formulated as piecewise uniform. The goodness of fit achieved in the model was low. Similarly, Pérez-Zuriaga et al. (2013) measured deceleration on tangent-to-curve transitions. They used linear regression to model the relationship between the radius of the oncoming curve and the deceleration rate developed. The values measured from individual speed profiles ranged from 0.4 m/s^2 (at the smoother curves), to 1.7 m/s^2 (at the sharpest curves). Perco et al. (2012) modelled the operating speed profile approaching and departing intersections. The results showed that the 85-th percentile acceleration rate was 0.7 m/s^2 whereas the counterpart deceleration rate was -1.14 m/s^2 . Rocci (1998) elaborated a mathematical model of the maximum acceleration performed by a vehicle as a function of

engine thrust and grade. However, very seldom do drivers exploit the maximum vehicle performance, thus this model is of little utility for sketching operating speed profiles. According to the TWOPAS model vehicle acceleration rates on level terrain vary linearly, but inversely, with speed (Fitzpatrick et al., 2000a; St John and Kobett, 1978). To account for grade effects, another term must be added to the performance equation, which reads:

$$a(v) = a_0 \cdot \left(1 + \frac{v}{v_{\max}} \right) + 9.81 \cdot G \quad (2.25)$$

where:

- a = vehicle acceleration at speed v , m/s^2 ;
- a_0 = highest acceleration achieved by the vehicle at zero speed, m/s^2 ;
- v = vehicle speed, m/s ;
- $v_{\max}$ = maximum speed attainable by the vehicle on level terrain, m/s ;
- G = grade, m/m .

As abovementioned, drivers do not usually fully utilize the acceleration capabilities, so reduction coefficients are used in the TWOPAS model. It is applied sequentially over a 1-s interval.

Concerning the location of the speed transitions, the model hypothesis of different authors do not always coincide. Whereas some authors assume that the transition occurs on exclusively tangents, other authors state that they take place within the curve, particularly on spirals (Pérez-Zuriaga et al., 2013). Islam and Seneviratne (1994) maintained that operating speed at starting points of curvature as well as at the tangency point differ significantly from operating speeds at middle point of the curve.

Figure 2.27 shows possible speed profiles between successive curves. Case a) occurs when it is possible to reach and maintain desired speed (dashed blue line) on the tangent between two curves. The limit case is presented in b), where the driver achieves desired speed on tangent and immediately slows down again. In c) a maximum operating speed is reached below desired speed. In d) if bends are close, forced deceleration (red line) may be assumed if the oncoming curve is not visible from the previous one. Otherwise, deceleration would start before the end of the curve is met.

2.6. Highway safety

A traffic accident is defined as an unforeseeable event that alters normal behavior of things and causes some damage, in which a moving vehicle is implied and takes place in the public road network. Accident risk constitutes an unavoidable liability to highway operation. The concept of risk in highway safety includes exposure, probability and consequences. The exposure can be quantified as the overall time (or times) along which users are in a potentially hazardous situation. The probability quantifies the chances that a failure in the system is produced under certain conditions. Finally, the consequences measure the severity of the potential failure (De Leur and Sayed, 2002).

Figure 2.27. Possible transition speed profiles between successive curves (Castro et al., 2008b).

Accident locations are not scattered completely random throughout the highway network, but somewhat distributed according to certain patterns which may be figured out. The disclosure of such patterns would prove the influence of highway design aspects on safety, supporting the importance of highway safety research.

2.6.1. Safety factors

The occurrence of traffic accidents may be attributed to the sum of several concurrent factors related to users, vehicles and infrastructure. Although in a high percentage of the accidents, one of the determinant factors is believed to be human error (Sabey and Staughton, 1975); the estimated shares of the underlying factors involved largely vary depending on the study looked up. Moreover, the enhancement of the vehicle features and the highway can contribute to the reduction of hazardous events and, therefore, the number and severity of accidents.

The influence exerted by the highway features on accidents is not always due to a single factor, but the combination of several or variations of those along the section considered. For example, double carriageways lead to a significant accident reduction, especially where entrances are restricted, since the regular accident type changes, disappearing head-on collisions and angle collisions.

The major relevant factors related to geometric design are cross section, horizontal and vertical alignments. First, lane width, shoulders and clearances exert a significant influence on safety. Also, medians are important on double carriageways. Overall, the average frequency of accidents is higher in curves with smaller radius. Where those curves overlap steep grades, the influence may become even greater. Moreover, intersections and junctions are conflict-prone spots. Therefore these spots have higher accident rates than other parts of the network.

The importance of pavement maintenance conditions comes from skid resistance when the surface is wet, especially on those spots where drivers must modify their speed or trajectory. The excessive roughness may cause an increase in collision risk since this fault makes driving more difficult. However, this roughness encourages drivers to reduce their speed, a fact that alleviates the increased risk of the aforementioned driving difficulty. It has actually been observed that resurfacing works, not accompanied by upgrades in the alignment or roadsides, led to higher accident rates owing to the increased comfort and therefore speed of drivers (Pardillo, 2004).

Roadside design and roadside safety devices are other important factors that can contribute to the mitigation of accident consequences. In relation to roadsides, highway legibility is the degree in which highway signs and devices contribute to minimize breaches of driver expectations and help in avoiding or mitigating accident consequences when the driver loses control over the vehicle.

The driver speed plays an important role both on frequency and severity of traffic accidents for several reasons. First, the distance travelled by a vehicle while the driver reacts to any variation in circumstances is greater. At a higher speed, the chances of recovering from a loss of vehicle control reduce. Especially on curves and intersections, performing successful evasive maneuvers turns out less feasible. Finally, accident severity increases as an impact, either against a mobile or static obstacle, is produced at higher speeds, or even in the case of vehicle rollover. The final impact speed depends greatly on the initial speed and thus, the probability of death.

The effect of traffic volume as a measure of exposure is clearly relevant as the number of accidents increases for higher traffic volumes. Moreover, the effect of traffic volume on accident rates varies among different highways. In two-lane rural highways, while the accident rate remains relatively constant between 4,000 and 12,000 vehicles per day, it raises sharply when volume decreases below 2,000 and increases steadily above 12,000 vehicles per day (Pardillo, 2004). In addition, the type of accidents changes as the volume varies. Single-vehicle accidents prevail where reduced traffic flows occur, while, as the volume increases, multiple-vehicle crashes are more frequent (Pardillo, 2004).

The influence of weather conditions of the area where facilities are located cannot be dismissed when studying highway safety, to the extent that it may cause specific safety issues. On the one hand, the presence of ice or wet pavement reduces significantly the adherence between tires and pavement. Heavy rain and fog, on the other hand, reduce sight distance drastically. Finally, frequent wind gusts may compromise safety for vehicles if their stability and maneuverability is impeded.

For a detailed study of the conditions surrounding each particular accident, statistical reports of the traffic authorities usually comprise the circumstances listed below (Kraemer et al., 2009):

- Location: milestone, driving direction, town.
- Day and time.
- Vehicles involved: number of vehicles involved, type of vehicles involved, pedestrians, etc.
- Consequences: number of injured, killed, property damage.

- Facility data: type, cross section, protection devices, signs, road markings, intersection, driveway, etc.
- Type of accident: head-on collisions, side and angle impacts, rear-end collisions, collisions involving pedestrians or cyclists, off-road excursions, etc.
- Lighting conditions: daylight, nighttime with and without lighting.
- Weather factors: rain, snow, fog.
- Pavement conditions: dry, wet, iced pavement and with snow.
- Drivers' data: drug consumption, use of seatbelt.

Figure 2.28. Variation of hazard index depending on traffic volume (Pardillo, 2004).

The classification of accidents according to these features allows handling disaggregated data to research the factors involved in accidents.

2.6.2. Accident rates

When planning highway safety policies, specific accident rates reflecting the risk level of accident occurrence along a particular highway section are used. These rates quantify the ratio between accidents with casualties, accidents with fatalities and accident exposure risk, measured along the length of the section. These rates, which are often used in highway safety programs, are defined as follows (Kraemer et al., 2009):

- Hazard index: Number of accidents with casualties / 10^8 veh. km
- Accident severity index: Number of accidents with fatalities / 10^8 veh. km
- Accident severity index: Number of fatalities / 100 accidents with casualties.

Accident databases in most developed countries usually record collisions that involve casualties. The extension of property-damage-only accidents is not enough to enable its use as a statistical

study database, even though there is a legal obligation to report it to both the authorities and the insurance companies. Hence researchers focus their analyses on accidents with casualties.

2.6.3. Highway operation data

Besides accident data, traffic volume information is necessary for highway safety research. The most used traffic data are annual average daily traffic (AADT) and the share of heavy vehicles. They determine the traffic flow and consequently influence the frequency and severity of accidents. These data are usually available from traffic count programs carried out by most highway administrations.

In addition, operating speeds are relevant data. The operating speed was addressed in section 2.5 on the theoretical level and under the assumption of free flow. However, the research may focus on actual flow conditions. Thus the speed measurement may require a large and reliable monitoring campaign along the network to be studied.

Inventories comprise the highway features over different design aspects, namely pavement, alignments and geometrics, signaling devices and protection structures. In this respect, the entry-into-service date of each section must be updated every time the characteristics, geometrics and features are altered owing to resurfacing, restoration and rehabilitation (RRR), or if the section undergoes major upgrade. In order to avoid the bias derived from a lack of such data in the records, some authors recommend that accident models do not cover periods longer than 3 to 5 years (Kraemer et al., 2009).

2.6.4. Statistical analysis of accident data

In order to set the adequate statistical method for the analysis of accident data, three important principles must be beard in mind. First, the probability distribution of collisions and the accident indices do not fit a normal distribution. Next, the distinction between the effect of the different concurrent factors affecting safety over time and the treatment applied to the accident occurrence is necessary. Finally, it is not always possible to develop statistical models of accident frequency from unbiased random samples (Pardillo, 2004). To achieve reliable results, it is necessary to rigorously evaluate which statistical procedure adapts better to the nature of the data and the initial hypothesis.

The two sources of error in statistics are bias and variance. Results are affected by bias if the expected value differs from the true underlying quantitative parameter being estimated. It usually arises when erroneous assumptions are made. The variance measures how far a set of random values are spread out from their mean.

In addition, the usual difficulties to determine the exact location where an accident took place and the actual location variability might make accident frequency unrepresentative when studied over short segments. Several authors recommended the consideration of a segment length no shorter than 300 m (AASHTO, 2010; Pardillo, 2004; Sacchi et al., 2012).

Besides accident rates, aggregated cost rates can be modelled using linear regression. For example, the German recommendation for safety analysis of road networks (FGSV, 2003)

provide the basis for accident cost evaluation and statistical modelling. Also, the Highway Safety Manual (AASHTO, 2010) provides a method to estimate crash cost by crash type.

Safety studies usually encompass the effect on traffic safety of geometric and operational variables such as cross-sectional elements, curve radius, CCR, degree of curve, length of curve, curve radii ratio, spiral features, grade, sight distance, traffic volume, design speed and operating speed. Some of these features refer to sections whilst some others apply to sites only. These parameters are commonly chosen for analysis because they are likely to exhibit a measurable influence on traffic safety, they can easily be measured, and previous accident research studies have found statistically measurable impacts of these parameters on traffic safety.

2.6.4.1. Statistical distribution of accident frequency

Both the frequency and the location of traffic accidents are subject to variations over time. From a statistical point of view, they may therefore be contemplated as discrete random events, independent among them and of rare occurrence (Pardillo, 2004). Thus, the usual approach to account for these variations is to consider the number of accidents occurred at a specific location over one year as a discrete random variable.

A generalized linear model (GLM) is a flexible generalization of ordinary linear regression that allows for independent variables that have error distribution models other than a normal distribution. They aim at finding the relationship between the dependent variable, in this case accident frequency, and one or several independent variables via a link function.

2.6.4.1.1. Poisson distribution

The Poisson distribution is the most common statistical model applicable to discrete random events occurring during a fixed period of time. Given the mean value $\lambda \in \mathbb{N}$, the probability of occurrence of $k \in \mathbb{N}$ events over a certain time period is given by the following expression:

$$P(\xi = k) = \frac{\lambda^k \cdot e^{-\lambda}}{k!} \quad (2.26)$$

The variance equals the mean k , therefore the standard deviation is $\sqrt{k}$. This distribution presents skewness and its shape depends upon k : if $k < 1$ mode is 0, whereas if $k > 1$ the probability increases up to $k-1$ and decreases from k . When the mean value is little, the standard-deviation-to-mean ratio is high, increasing the likelihood of occurring values considerably different from the mean. Consequently, precautions should be taken in accident records subject to such a random oscillation. An alternative procedure of analysis must be considered otherwise (Pardillo, 2004).

2.6.4.1.2. Negative Binomial distribution

A common analysis error is a result of failing to satisfy the property of the Poisson distribution that restricts the mean and variance to be equal (Washington et al., 2010). If this equality does not hold, the data are said to be underdispersed or overdispersed. The negative binomial distribution reads:

$$P(\xi = k) = \binom{k+r-1}{k} (1-p)^r \cdot p^k \quad (2.27)$$

In a sequence of independent Bernoulli trials (years in accident analysis), $k \in \mathbb{N}$ is the number of trials whereas $p \in [0, 1]$ is the probability of an event occurring in each trial and $r \in \mathbb{N}$ is the number of failed trials before the event occurs. The mean and the variance of this distribution read:

$$E(\xi) = \frac{pr}{1-p} \quad V(\xi) = \frac{pr}{(1-p)^2} \quad (2.28)$$

Several authors found that this distribution predicted well accident occurrence on highway segments (Ibrahim and Sayed, 2011; You and Easa, 2009).

2.6.4.1.3. Zero-inflated Poisson distribution

The zero-inflated distribution, which allows for frequent zero-valued observations, helps in modelling the accident frequency in the terms described in this section. It concerns a random event containing excess zero-count data in unit time. It is first governed by a binary distribution that generates zeros. The second process is determined by a regular Poisson distribution that generates counts, some of which may be zero. The two model components are defined as follows:

$$P(\xi = 0) = p + (1-p)e^{-\lambda} \quad (2.29)$$

$$P(\xi = k) = (1-p) \frac{\lambda^k \cdot e^{-\lambda}}{k!} \quad \text{if } k > 0 \quad (2.30)$$

where p represents the probability of extra zeros and k is the expected Poisson count. The mean and the variance equal:

$$E(\xi) = (1-p)\lambda \quad V(\xi) = (1-p)(1-\lambda \cdot p)\lambda \quad (2.31)$$

Easa and You (2009) used this distribution to predict accident frequency on curves.

2.6.4.1.4. Zero-inflated negative binomial distribution

This distribution also increases the probability of obtaining zeros compared to the binomial distribution. This distribution can better predict collision frequency on sections where the likelihood of a vehicle accident occurring may be extremely small.

$$P(\xi = 0) = p + (1-p) \left(1 + \frac{\lambda}{\tau}\right)^{-\tau} \quad (2.32)$$

$$P(\xi = k) = (1-p) \frac{(k-\tau-1)!}{k! \cdot (\tau-1)!} \left(1 + \frac{\lambda}{\tau}\right)^{-\tau} \left(1 + \frac{\tau}{\lambda}\right)^{-k} \quad \text{if } k > 0 \quad (2.33)$$

The new parameter τ is the shape parameter which quantifies the amount of overdispersion. The mean and the variance are given by:

$$E(\xi) = \lambda \quad V(\xi) = (1-p) \left(1 + \lambda \cdot p + \frac{\lambda}{\tau} \right) \cdot \lambda \quad (2.34)$$

Easa and You (2009) used this distribution to predict accident frequency on curves.

2.6.4.1.5. Chi-square test

To assess the goodness of fit of the abovementioned models, the Pearson χ^2 statistic is utilized. For each predicted value y_i , a real value μ_i is associated according to the general expression that follows:

$$\chi^2 = \sum_{i=1}^n \frac{(y_i - \mu_i)^2}{\mu_i} \quad (2.35)$$

It is asymptotically χ^2 distributed with $n-p$ degrees of freedom, where n is the sample size and p is the number of parameters of the selected statistical distribution. A high value or close to 0 value of the χ^2 statistic would indicate that the model has poor prediction performance.

2.6.4.1.6. Confidence levels in statistical tests

The statistical tests applied in accident data analysis aim to estimate population parameters from a given sample. A confidence level refers to the percentage of all possible samples that can be expected to include the true population parameter. The resulting confidence and significance levels from those statistical tests can be associated to a practical extent to the subjective interpretations that are reflected in the following table.

Table 2.5. Interpretation of confidence and significance level in statistics (Pardillo, 2004).

Significance	Confidence level	Subjective interpretation
1 %	99 %	Highly acceptable
5 %	95 %	Acceptable
10 %	90 %	Indicative
20 %	80 %	Weakly indicative

2.6.4.2. Accident analysis techniques

In order to analyze the effect of the highway features on accident occurrence, two types of statistical methods are used: before-after studies and regression models.

Before-after studies consist of comparing the evolution of accident frequencies on sites where the facility features have been altered, over periods of few years before and after the changes. The motivation of such studies is the existence of a collision modification factor (CMF) that is likely to cause a change in the accident patterns. A CMF is a multiplicative factor used to reflect the expected change in safety performance associated with a change in highway design or a

traffic control feature. Oftentimes the results of such studies may be questionable due to the failure to consider 'regression to the mean' effects, and/or insufficient data.

The second method consist of examining accident data, traffic volume and road features of a broad sample of sites over a period of study through regression analysis. This statistical procedure aims at obtaining functions that link accident frequency and road features.

2.6.4.2.1. Regression models

By applying regression techniques, it is possible to estimate the relationship between accident rates and the road features (section 2.6.2). Accident records, traffic volume and road feature inventories provide models with data. However, the application of these techniques involves difficulties owing to the randomness of accident occurrence as well as the interaction of a great amount of factors. For this reason, low significance levels are frequent in the outcome (Pardillo, 2004).

Generally, the initial hypothesis assumes that the frequency of accidents at each site presents a random variation with a distribution of mean λ (Poisson, Negative binomial, etc.). The multivariate analysis is applied to estimate the value of λ in each section of the network as a function of a set of variables x_i that characterize the road features and the corresponding calibration parameters β_i

$$\lambda = f(\{x_i\}, \{\beta_i\}) \quad (2.36)$$

The eventual model must comply with three requirements:

- Regression coefficients of each variable must be statistically significant. In other words, the null hypothesis that the coefficient equals zero should be able to be rejected. Otherwise, non-statistically-significant variables will have to be removed from the model. The fact that autocorrelation among variables may distort the model output has to be borne in mind.
- The sign and magnitude of coefficients should explain effectively the expected effect on accident occurrence.
- The goodness of fit should be checked through adequate statistical tests to the model.

2.6.4.2.2. Safety performance functions

A safety performance function (SPF) is an equation in the shape of a generalized linear model (GLM) that aims to predict the average number of accidents per year at a location as a function of exposure and, in some cases, roadway or intersection characteristics (e.g. number of lanes, roadway width, traffic control, etc.) (AASHTO, 2010). For highway segments, exposure is represented by the segment length and the AADT. A GLM is a flexible generalization of linear regression. The general expression of these models in accident analysis is exhibited in Equation (2.37).

$$\lambda = \alpha_0 \cdot \prod_{i=1}^m x_i^{\alpha_i} \cdot \exp \left(\beta_0 + \sum_{j=1}^n \beta_j \cdot y_j \right) \quad (2.37)$$

where:

- x_i = exposure variables;
- α_0, α_i = adjusted model coefficients of exposure variables;
- y_i = additional variables;
- β_0, β_i = adjusted model coefficients of additional variables;

This type of model guarantees that the predicted value of the mean accident frequency per time unit is an integer number. To devise the statistical model, it must be assumed that collisions are independent events of one another whilst the variables are independent and constant over the period of study.

2.6.5. Research on sight distance and accidents on highways

Much research has been performed aiming at quantifying the effect that the geometric design of highways exerts on traffic safety. Research on highway safety should be supported by the real accident occurrence (Heine, 2013). Hassan et al. (1998b) reported that higher accident rates manifest in sections where alignment is more deficient. Sight distance, as a fundamental element of geometric design, takes part on the influence on safety. According to Babkow (1975), an insufficient ASD was the foremost cause of as many as 8 to 10% of accidents in the former Soviet Union. Also, an insufficient ASD was found to be an important cause of accidents in China (Zhang et al., 2008).

The effect of sight distance on safety has been addressed in several studies. Olson et al. (1984) compared accident occurrence between crest vertical curves where ASD is limited and control sections meeting the SSD standards in Michigan, concluding that there are significantly fewer accidents at sites where ASD is adequate. Sparks (1968) reported a negative correlation between ASD and accident rates in the United States. Sylianov (1973) and Kunze (1976) drew similar conclusions about highways in the former Soviet Union and Germany, respectively. More recently, Fambro et al. (1989) analyzed crash rates and ASD at crest vertical curves in Texas. They, conversely, concluded that limited ASD did not necessarily create safety issues. Where the degree of deficiency was marginal compared to the standardized ASD, it did not affect crash rates; however, once the ASD was noticeably shorter, it did affect accident occurrence. They did not provide, nonetheless, any measure of the sensitivity of crash frequency to the ASD decrease. Urbanik et al. (1989) addressed the effects of limited ASD on crest vertical curves also in Texas. Whereas the relationship between ASD on crest vertical curves on two-lane roadways and accidents was found difficult to quantify, intersections within the limited sight distance sections of crest vertical curves showed a statistically significant increase in accident rates. Moreover, Fitzpatrick et al. (2000b) found that limited ASD does not necessarily pose a safety problem. Hristov (2009) stated that the disagreeing results of many studies do not currently allow any final assessment of the question of whether an insufficient ASD has a decisive influence on traffic safety.

One of the main objectives in road design is to ensure high safety standards. This is normally achieved through a generous alignment, which promotes long ASD. As stated in section 2.2.2.1, the ASD must be sufficient at any point to exceed the required SSD. Glennon (1987) addressed the trade-off in enhancing visibility. He affirmed that alignment improvements are cost-effective

only on high-traffic volume highways because striking an object on the pavement surface is a very odd event. However, providing long enough ASD is helpful not only to avoid crashes onto unexpected items but also the correct appraisal of the course of the highway is strongly determined by the visibility conditions. Coburn (1962) reported that the number of accidents with injured and fatalities in curves decreased by 65% after improvements in visibility on British highways. Actually, Fink and Krammes (1995) found that ASD explains some of the variability in crash frequency accounting for the speed reduction when approaching curves. They assumed two different scenarios within the sample analyzed: the curve is noticeable or not from a long-enough distance ahead on the approach tangent. However, since sites were clustered into sight-distance intervals, much of the variability was removed. The same remark can be applied to the results of Krebs and Klöckner (1977). They analyzed 1,150 km of two-lane rural highways in the Federal Republic of Germany. It was found that the accident rate and, to a lesser extent, the accident cost rate decrease with increasing available sight distance. This decrease is observed to be particularly stronger in accidents rather than in costs. The accidents have significantly increased only when the ASD is less than 100 m (maybe influenced by passing accidents). Nonetheless, whether this relationship is caused by sight distance conditions or by geometric design, it is not clearly explained. The authors identified a link between increasing CCR and increasing accident occurrence. The influence of the curve radius on accident occurrence is more pronounced than that of ASD. Due to the demonstrated strong correlation between sight distance and curve radius both factors are strongly recommended to be considered for highway safety. The study of Krebs and Klöckner (1977) yielded the adjusted Equation (2.38), which predicts the accident rate (accidents per 10^6 veh·km) and the stated goodness of fit was $R^2 = 0.91$. The curve is plotted in Figure 2.29.

$$A_R = 7.391 - 4.07 \cdot \log(ASD) + 0.294 \cdot [\log(ASD)]^2 \quad (2.38)$$

Figure 2.29. Accident rate as a function of the ASD (Krebs and Klöckner, 1977).

Other researchers obtained interesting results using linear regression to predict accident rates. Silyanov (1973) yielded a regression equation (2.39) to describe the relationship between the accident rate and the ASD. The equation applies to distances ranging from 25 m to 800 m.

$$A_R = \frac{1}{0.2 + 1.11 \cdot 10^{-3} \cdot ASD + 9 \cdot 10^{-7} \cdot ASD^2} \quad (2.39)$$

Similarly, Bögel (1967) yielded an equation (2.40) for ASD between 50 m and 850 m.

$$A_R = 4.037 - 6.32 \cdot 10^{-3} \cdot ASD + 4.9 \cdot 10^{-6} \cdot ASD^2 \quad (2.40)$$

It concluded that the effect of lane width on accident rate is more evident on highways where it drops below 7 m. Hiersche (1968) stated that accident rate is directly linked to roadway width, to curvature change rate and, indirectly, to ASD, keeping in mind that a more generous geometric design implies longer ASD. He concluded that sight distance is by far the most important design and evaluation criterion for road design.

Meyer et al. (1961) found that 24% of all accidents on two-lane highways occur in connection with passing maneuvers and, in most cases, where ASD is not sufficient to allow passing maneuvers. Netzer (1966) came up with similar results, finding that the proportion of these type of accidents is 21%. Hammerschmidt (2008) studied a large two-lane highway network and found that accident rates increased as the roadway width decreases. They also increased as the curve radius or ASD decreased.

Bernhard (1999) examined rural highway sections with insufficient SSD in search of traffic safety problems. It was found that limited ASD is not an accident-causative factor itself in most cases. Also, after analyzing motorway sections, he concluded that there is no significant correlation between accident black spots and crests with limited ASD. Furthermore, Bernhard (1999) did not find radius decrease at horizontal curves to contribute significantly to increase accident rates on German motorways. Conversely, Steinauer et al. (2002) found correlation between ASD and accident rate reduction both at left and in right bends on the Bavarian motorways concerning driving accidents (in which the driver loses control of the vehicle). Indeed, the speeds driven on German motorways may require significantly longer SSD than the ASD, especially in small-radius left curves, where ASD might be limited by e.g. vegetation, anti-glare, guard rails or bridge piers (Steinauer and Mayer, 1999). In the event that required SSD could not be met, severe safety deficiencies in the inner lane in left curves may arise. Steinauer and Mayer (1999) define 'visibility accidents' as the accidents in the longitudinal movement where conflict situations arise such as rear-end collisions, jam collisions, lane-change collisions (side impacts), and other accidents with conflict situations involving mobile obstructions or broken-down vehicles. It was found that 60% of the accidents with injured and fatalities occurred in curves where the radius was less than 1050 m, which corresponds to 130 km/h at the 85-th percentile speed according to the German standard for motorways (FGSV, 2008b). A number of 298 accidents out of 2,563 could be attributed to poor visibility conditions, which means 11.6 %. The 85-th percentile speed was determined on inner lanes in wet pavement condition to carry out the experiment. The importance of the centripetal component of adherence is also claimed. From that research, it is however not clear what proportion of length represent these tight curves of the overall length of all curves of the examined motorway network. Therefore, the question of the systematic risk

owing to limited ASD could not be addressed adequately. Furthermore, no distinction was made in the investigation after accident in left or right curves. Also, the influence of restricted ASD in left bends was investigated by Krüger et al. (2004) on freeways. Forty tight left curves with a radius smaller than 1,500 m, where the ASD is known to be limited by the median barrier, were analyzed. Crash occurrence on these segments was compared to those on control sections: the opposite right-hand bends, as well as other four sections before and after the left and the right curves. It was found that, in almost all cases considered at the left bends, where the SSD is shorter, there was a higher injury incidence connected to higher accident costs than that on the control sections. Further evaluations were made with regard to the effects of other individual parameters such as curve radius, degree of reduction of the ASD, AADT, and speed limit. The left curves were compared with the paired right curves of the opposite direction. These analyses indicated that the left curves, in almost all cases, involved a higher accident risk than the right curves.

Hristov (2009) aimed to provide insight on the relation between driving behavior and accident occurrence on motorways. The ASD and SSD were compared at locations where rear-end, head-on and one-car (against-object collisions and off-road excursions) accidents took place. The author did not find a clear trend linking sight distance and accident rates in the selected accident types.

The heterogeneity of these studies must be highlighted. All these accident prediction models and statistical studies taking ASD into account were developed using different ASD estimation methods. In many cases 2D methods and 3D methods in other cases. As the case may be, it is not clear whether all highway features and elements by the roadsides were regarded.

Apart from that, visibility conditions are related to 3D alignment coordination and therefore could be connected in some way to highway aesthetics. Hence Lamm et al. (1999) assert that an aesthetically pleasant appearance is greatly appraised by drivers in terms of safety and service. You and Easa (2009) elaborated collision prediction models on horizontal tangents, taking 3D alignment coordination into account, on two-lane rural highways in the Washington State. They distinguished models for several combinations of vertical alignments on horizontal tangents. On horizontal tangents combined with multiple vertical curves, where cases of sight-hidden dips occur, the change of vertical curvature was found to be significant, yet its influence was very low. Horizontal tangents combined with sag vertical curves, with crest vertical curves and with different constant grades were also considered, where other geometric design issues were exposed. In parallel, Easa and You (2009) developed collision prediction models on horizontal curves on the same highways. The same combinations with vertical alignments were regarded and the most significant accident predictors were the degree of curvature, curve length, roadway width, access density and AADT.

2.6.6. Safety-related shortcomings in geometric design

When a bend in the horizontal projection cannot be foreseen, a concealed start of curve is produced. Concealed starts of curves are potentially hazardous for drivers. For that reason they are considered a safety-related shortcoming in the spatial alignment (FGSV, 2008a). Hidden dips are also contemplated in such a guide despite the fact Weise et al. (2002) did not find significant relation with accident occurrence.

Kühn (2013) affirms that an alignment composed of a sequence of SSE can reduce to a minimum the number of shortcomings in 3D alignment. Yet SSE were meant to reduce the chances of producing concealed start of bends, no clear relation to hidden dips seems to exist in this regard.

2.6.7. Safety evaluation of highways

Researchers acknowledge the importance of adopting proactive measures to reduce accident frequency on roads. Based on accident occurrence, reactive measures not only are subject to great randomness and to the quality of accident records but do not permit to take action before casualties are produced either. Hence surrogate techniques should be developed in this respect.

The driver-based assessment facilitates a more detailed analysis of sections which is not available on police reports of accidents (De Leur and Sayed, 2002). Based on this, De Leur and Sayed (2002) developed a road safety risk index based on the subjective assessment of a set of parameters in order to avoid relying on the quality of accident databases. The road geometry and the ASD at intersections were some of the factors therein considered. To provide a measure of risk, different road features were incorporated, quantifying exposure, probability and consequence. Such a technique was tested and validated in a real corridor in Canada. Similarly, Cafiso et al. (2007) validated a safety index incorporating quantitative measures of exposure, probability and consequences of accidents.

Traffic conflicts are more frequent than collisions. Thus this events represent a potential database for road safety analysis. Current automated data extraction based on computer vision techniques are applied to video recordings in order to facilitate the examination of conflict occurrence (Ismail, 2010).

Finally, theoretical developments can be applied to predict collision prone locations. This is the case of reliability analysis, a technique that is explained in the next section.

2.7. Reliability analysis

In recent practice, CMFs are typically utilized to evaluate the impact on safety associated with changes in road features or traffic control (AASHTO, 2010). However, in many situations, there are no CMFs available to predict the safety impact of particular road features such as sight distance restriction on horizontal curves or crests. Recently, reliability analysis has been advocated as an effective approach to account for uncertainty in the geometric design process and to evaluate the risk associated with a specific design. In this approach, the design parameters are considered as random variables expressed in terms of their probability distributions as opposed to single value estimations in the deterministic approach. Design equations are represented as limit-state functions (LSF). The LSF represents the difference between the supply and the demand in a system. If the demand exceeds the supply (i.e. $LSF < 0$), the design is considered to be failed or not complied with the standard design requirements. Reliability theory is used to evaluate the probability of failure, which is associated with a measure of probability that the demand exceeds the supply or that an undesired event exceeds a certain threshold.

2.7.1. Reliability theory

The term reliability refers to the complement of the failure probability (Equation (2.41)). In the reliability analysis, the term probability of failure represents the probability of undesired event exceeding a certain threshold. In road design, researchers have proposed the use of the probability of non-compliance (P_{nc}) to label the probability of a design that does not meet standard (Essa et al., 2016; Hussein et al., 2014; Richl and Sayed, 2006).

$$Reliability = 1 - P_{nc} \quad (2.41)$$

The reliability problem has two components: random variables, which describe the uncertainty, and LSF, which defines the failure mode. The first step is defining a LSF; denoted by $g(x)$; which describes what is considered to be noncompliance, where x is a vector of random input variables (Haukaas, 2012a) as expressed in Equation (2.42). Generally, the LSF is represented as a balance between supply and demand (Figure 2.30). For example, the supply is the ASD, and the demand is the required SSD. The P_{nc} is given by Equation (2.43), in which $f(x)$ is the joint probability distribution function for random variables (Haukaas, 2012a).

$$g(x) = S(x) - D(x) \Leftrightarrow \text{Failure or noncompliance} \quad (2.42)$$

$$P_{nc} = P[g(x) \leq 0] = \iint_{g(x) \leq 0} \dots \int f(x) dx \quad (2.43)$$

Figure 2.30. Basic components of a LSF.

The measure of safety in a stochastic system can be expressed as the difference between the expected value for supply and the expected value for demand (El-Bassiouni and Sayed, 2010):

$$M_s = E(S) - E(D) \quad (2.44)$$

And standard deviation is defined similarly:

$$\sigma_{SD} = \sqrt{\sigma_S^2 - \sigma_D^2} \quad (2.45)$$

The safety level can be quantified by the reliability index. Using Equations (2.44) and (2.45), it can be defined as follows:

$$\beta = \frac{M_S}{\sigma_{SD}} \quad (2.46)$$

2.7.1.1. Reliability methods

In most cases, the integral in Equation (2.43) cannot be solved analytically. Therefore, various reliability methods may be used to provide approximate solutions, which are summarized below.

The mean-value first-order second-moment (MVFOSM) method considers only second-moment information of the random variables together with a first-order approximation of the LSF about the mean of the random variables. This method is the simplest one, but has three main disadvantages: first, it may lead to different results for equivalent LSF's, known as the invariance problem (Haukaas, 2012b); the linearization of a non-linear LSF may lead to a biased estimation of the P_{nc} ; and the distribution type of the random variables is systematically ignored.

The first order reliability method (FORM) remedies the invariance problem of MVFOSM and incorporates the distribution type of the random variables. Nevertheless, FORM maintains the linearization of the LSF and may therefore lead to inaccuracy when the LSF is nonlinear (Haukaas, 2012a). Conversely, the second order reliability method (SORM) approximates the LSF by a second-order function, overcoming the possible inaccuracies when the LSF is nonlinear, yet more computational capacity is required compared to the FORM (Haukaas, 2012c).

Finally different sampling schemes are available for reliability analysis. They evaluate the LSF and produce an approximate value of the P_{nc} (Haukaas, 2012d). The simplest and most popular is called Monte Carlo.

2.7.2. Reliability applications in transportation

The reliability analysis technique has been successfully applied in many fields of engineering. It contemplates, as opposed to deterministic design, the probabilistic nature of the variables in engineering systems. The deterministic approach is followed by design standards on highway geometry. Two important drawbacks have, nonetheless, been reported on such methodology (Ismail and Sayed, 2010). First, the selection of the design values is not founded on definitive safety levels. Design parameters are typically selected at conservative percentile values drawn from their respective distributions among the general population of road users. Thus the safety margin of the design output is largely unknown. Second, there is little knowledge on the safety implications of deviating from the design standards, as occurs when providing ASD less than that required SSD by the pertinent standard. In other words, a slight violation of the SSD requirement is tantamount to the most blatant visual obstruction, being both unacceptable.

Several studies applied reliability analysis to research problems in the transportation engineering field. Ben-Akiva et al. (1985) proposed a probabilistic cost-based highway design. Faghri and Demetsky (1988) used a probabilistic approach to evaluate the limitation in ASD at road and railway grade crossings. Easa (1994b) proposed a reliability-based design of sight distance at railroad grade crossings. Also, Easa (2000) applied the MVOSFM reliability method

in order to evaluate sight distance at intersections. Navin (1990) proposed a general method to specify the design parameters based on factors that indicate the importance of a road. Moreover, Navin (1992) proposed a method to estimate the margin of safety and reliability index for isolated highway components, where SSD is analyzed to validate the method.

More specific research has also been carried out in highway geometric design. For example, Echaveguren et al. (2005) proposed a methodology based on reliability theory to determine the margin of safety of an existing horizontal curve. Richl and Sayed (2006) applied the FORM to evaluate the safety risk of narrow medians combined with tight horizontal curves. With regard to sampling methods, Khoury and Hobeika (2007) used Monte Carlo simulation to estimate the risk level of passing maneuvers based on the probability of inadequate PSD and the consequences of a collision. Sarhan and Hassan (2008) utilized Monte Carlo simulation to estimate the P_{nc} in 3D SSD, on overlapped horizontal and vertical curves.

Reliability analysis has been utilized in guide calibration in order to yield consistent safety levels for design. Ismail and Sayed (2009) introduced a general framework for calibrating standard design models and determining target value of design safety. Also, Ismail and Sayed (2010) investigated the safety implications of sight distance limitation on road segments, and the risk associated with deviation from standards. Furthermore, Ismail and Sayed (2012) presented a decision mechanism that enables the efficient use of available right-of-way for new highway construction with restricted sight distance to minimize the overall risk of the design. Ibrahim et al. (2012) presented a methodology to select a suitable combination of cross-section elements with restricted sight distance to reduce collision risk and yield consistent risk levels. Hussein et al. (2014) investigated the calibration of geometric design models to produce consistent risk levels. Llorca et al. (2014b) performed a reliability analysis of PSD based on maneuver observation in two-lane rural roads.

Himes and Donnel (2014) developed a probabilistic approach to the design of horizontal curves considering the effects of wet pavements and tire characteristics for passenger cars and heavy trucks. Musunuru and Porter (2014) applied reliability analysis to evaluate the probability distribution of operational performance associated with decisions made to achieve a design level of service for number of lanes on a freeway. You et al. (2012) used different performance functions for calculating the probability of vehicle failure modes, namely skidding and rollover, at horizontal curves. You and Sun (2013) established a dynamic simulation model, considering three-dimensional alignment, for reliability analysis of vehicle stability on the combined horizontal and vertical curve. Shin and Lee (2013) presented a reliability-based analysis to assess vehicle safety on horizontal curves based on vehicle dynamics on windy environments. Essa et al. (2016) proposed the use of system reliability-based design of horizontal curves using two failure modes, namely SSD and vehicle skidding.

2.8. Alignment data extraction

The alignment extraction of an in-service highway is a necessary process to evaluate the geometric design of highways. It can be considered as a reverse-engineering technique that consists of obtaining the parameters of the theoretical alignment elements that best adjust the geometric layout of an existing highway. Generally, as-built drawings from which highway alignments can be derived are unavailable. Alignment restoration enables engineers to derive

the highway centerline, which is a basic input for sight distance analysis and is even more crucial for 3D alignment coordination studies. Hence it is necessary to recreate in an efficient, cost effective and practical way the alignment when highway geometric design is to be analyzed.

As a first step towards alignment restoration, the adjustment of highway centerlines has been tackled by several authors using cubic splines on horizontal alignment (Castro et al., 2006b; Garach et al., 2014a) and on 3D data (Liao et al., 2010). Goodness of odd-degree piecewise-defined functions (i.e. continuously differentiability of class C^2 , smoothness and simplicity) make them suitable for the centerline adjustment.

Next, featuring the geometry of the road must be pursued. A frequent data source for alignment restoration is GPS dataset extraction obtained from a section car drive (Cafiso and Graziano, 2008; Castro et al., 2006b; Imran et al., 2006). Software implemented on GIS was used to deduce alignment elements in curves with no spirals from GPS vehicle path datasets (Imran et al., 2006). Using GPS also, Pérez Zuriaga et al. (2010) tracked the average vehicle path as the average path of several vehicles and defined thus the “operational alignment”. Moreover, newer technologies can be applied to this assignment. Aided by GIS, a method was proposed for extracting the elements of simple circular curves and reverse curves using satellite imagery (Easa et al., 2007), which was further enhanced shortly after (Dong et al., 2007). Li et al. (2015) used GIS to preliminarily detect horizontal curves on low volume rural highways from polyline feature classes. Tsai et al. (2010) computed roadway curvature values from conventional digital cameras mounted on a car. Cai and Rasdorf (2008) developed an algorithm to detect road centerlines from LiDAR clouds of points.

Alignment segmentation can be man-made performed. Nonetheless, it is a labor-intensive work and does not guarantee a desirable accuracy, which is needed for certain studies. It is convenient therefore, to automatize the restoration process. Garach et al. (2014b) proposed a segmentation method for horizontal alignment based on curvature data. Good accuracy on radii estimation was achieved, albeit low-deflection-angle curves or complex curve sequences can be hardly detected using such a methodology. Several authors proposed analytical methodologies to deduce the theoretical components of the horizontal alignment of highways using azimuth datasets (Camacho-Torregrosa et al., 2015; Gikas and Stratakis, 2012; Karamanou et al., 2009). In the procedure proposed by Karamanou et al. (2009), it is the user who must define the stitching points between elements of the alignment. Gikas and Stratakis (2012) proposed an arbitrarily chosen small threshold value of curvature to set the segmentation. They first filtered the noise of the azimuth dataset. Conversely, Camacho-Torregrosa et al. (2015) proposed in their methodology that the user must mark one spot on each tangent only, whilst a heuristic algorithm seeks for the sequence of curves that best fit the azimuth dataset between two marked spots. The major contribution of this methodology is the accuracy achieved in parameter estimation as it eludes the effect of data noise on results. This facilitates an accurate detection of spirals, low-deflection-angle horizontal curves as well as an appropriate procedure to tackle the restoration of complex successions of horizontal curves. The main drawback, nonetheless, is the accumulated error on the horizontal projection since it applies an azimuth-based optimization criterion and not based on the (x, y) coordinates. This makes the methodology not suitable for sight distance analysis nor for alignment coordination studies, where planimetric precision is fundamental.

Commercial software developers have also broached this issue. For example, CARD/1 (2014) recently incorporated an automatic alignment generation module. Based on Powell's optimization method, Trivium uses the curvature and azimuth datasets to deduce the alignment data, the user being allowed to select where the stitching points are likely to be located in order to enhance the computational performance (Puy, 2014). Other commercial solutions were developed to reproduce the best fit alignment of existing facilities (Pettus, 2015; Shihundu, 2016). It should be noted that, besides its possible use to fully determine the highway centerline from a surveyed set of points, these tools can be exploited to generate brand new highway alignments very quickly given a set of waypoints or geometric constraints.

Tarel et al. (2007) designed a fully-automated algorithm for vertical alignment restoration derived from rectified stereo images. Marinelli et al. (2017) proposed a methodology to reproduce the existing horizontal alignments based on pixel recognition in georeferenced images from onboard cameras, comparing the results obtained to those of other techniques. Similarly to these procedures in the horizontal alignment, Karamanou et al. (2009) proposed a methodology to restore vertical alignment based on slope datasets in which the stitching points between elements are marked by the user.

As a concluding remark on this issue, Bird and Hashim (2005) reported that many rural single carriageways follow historical alignments as they predate design standards. Consequently, the estimation of a theoretical alignment may not fit well the actual geometry of the highway depending on the cases.

2.9. Geographic information systems in highway research

Transportation issues lend themselves for the use of GIS for many reasons (Thill, 2000). The usefulness of GIS goes beyond organizing information in databases or displaying data on a map. GIS capabilities adjust well to the specific requirements of the transportation domain of application. Particularly, spatial analysis requires little effort thanks to the multiple tools available. Some examples have been enumerated over the present chapter showing the suitability of GIS. These applications included sight distance computation and analysis (Castro et al., 2011; Castro et al., 2014a), alignment data extraction (Li et al., 2015), and operating speed and highway design consistency analysis (Castro et al., 2008b). The potential of GISs in spatial analyses and the increasing number of road inventories supported by a GIS, including traffic data and crash rates, suggest that it can be useful for safety studies. In addition, the use of GIS facilitates finding accident patterns in traffic safety analysis (Steenberghen et al., 2004).

2.10. Review summary

This review highlighted that sight distance has been acknowledged by researchers to be a fundamental highway design input. Adequate sight distance conditions must be provided not only at the infrastructure delivery but also all over its service life. One of the problems arises at the selection of the sight distance evaluation technique and the inputs utilized. Several interesting 3D sight distance estimation techniques have, nonetheless, been found in this review.

In the case of in-service highways, the first challenge faced by designers and researchers is the reliance of data. Moreover, the analysis of sight distance on in-service facilities should be capable of dealing with the lack of available or reliable alignment data. Consequently, the methodology to be used must be robust to whatever the data source is: construction project, inventories, field measurement data, etc.

The use of different terrain and landscape modelling techniques deserves particular attention. It is necessary that landscape features are contemplated in the sight distance calculation due to the evident sight restriction effect, especially where the horizontal projection of the highway is not straight. Moreover, the safety implications of these restrictions should be researched.

The relation between the ASD and safety is cause of discrepancies as shown in this literature review. First, some traffic safety studies estimate the ASD using archaic low-performance techniques such as conventional topographical measurements, others consider 2D methodologies which probably lead to biased results. Hence the disagreeing results with regard to safety. To overcome this issues, at least the two following facts must be taken into account. First, a reliable 3D sight distance evaluation method along with adequate inputs should be used to deliver an unbiased sight distance outcome. Second, an appropriate technique should be employed to evaluate sight distance requirements with regard to driving performance, it enabling the search of links between ASD and highway safety.

Reliability theory provides a framework to model the response of engineering systems. It has been applied successfully in many fields of engineering. Lately, transportation issues have been broached using this technique, including some related to geometric design. However, concerning the latter ones, no study contemplating 3D design was found. While the differences between 2D and 3D ASD estimation methods have been subject of analysis, the safety implications of the different sight distance estimation approaches are still unknown. Therefore, these potential safety issues need to be addressed.

Reliability analysis can also bridge the gap between highway design and safety by helping designers to cope with uncertainty and therefore define more consistent levels of design. There are no CMFs available to predict the safety impact of particular road features such as sight distance restriction on horizontal curves. In this respect, research of in-service facilities has deserved little attention so far. Therefore, the analysis of emergency stopping failure mode from current ASD estimation techniques results of particular interest, where reliability-based analysis can provide a surrogate measure for CMFs.

Owing to the subjectivity of visual perception, setting an appropriate methodology to analyze certain aspects of 3D alignment coordination is complicated. This is the reason why few features of the 3D alignment coordination of the highways are subject to quantitative analysis. Geometric design standards are essentially limited to descriptive recommendations where almost no parameter to assess spatial alignment shortcomings is available. Researchers have aimed at filling this gap with either cumbersome or ineffective procedures that resulted impractical.

According to this review, sight-hidden dips are among those shortcomings which entail a greater hazard, along with hidden beginning of curves, for the safe operation of highways. Sight-hidden dip parameters have been fairly characterized in literature: range, depth of the hidden section,

maximum length of the interim hidden section and distance to the end of the hidden section. The methods available based on line-of-sight loops are not able to discern whether the existence of concealed intermediate roadway areas are caused by the lack of lateral clearances on reverse curves in plan or, conversely, are due to a profile that adjusts to the terrain too strictly. In addition, sight-hidden dips have been studied from a theoretical point of view only, no real case analysis has been found.

The methodology to broach the alignment coordination presented by the German normative framework is a worldwide renowned reference. They propose a quantitative approach that is not found in any other normative framework. The Swiss and the Italian standards also offer guidelines to prevent hazardous sight-hidden dips, though they do not seem to be very effective. The rest of the instructions either just give a descriptive framework or even did not mention alignment coordination.

As occurred for sight distance, the analytical 2D approach on hidden dips is of limited utility for two reasons. First, not all the involved alignments before the crest and beyond the sag curve are contemplated. Second, sight-hidden dips on non-straight horizontal alignment cannot be analyzed. Therefore, it would be necessary to be able to broach the issue considering its 3D nature in an appropriate manner.

Finally, with regard to alignment data extraction, the use of heading to deduce alignments produces satisfactory results in most cases. However, not enough accurate results are produced by this technique itself in some other cases when the planimetric adjustment of the alignment outcome is overlooked. The combination of heading and position in planimetry would provide the best fit output. In addition, the recreation of compound curves remains the weakest aspect of alignment recreation techniques.

Chapter 3

METHODS AND MATERIALS

3.1. Computation of sight distance

3.1.1. Introduction

As it was stated in section 1.4, the main objective of this thesis is to devise a methodology to analyze 3D sight distance on highways. Prior to the analysis, a powerful and intuitive tool to compute sight distance is necessary for designers and researchers. In this sense, an application that exploits ArcGis for Desktop® capabilities was utilized to calculate 3D sight distance and aids in the analysis. In particular, ArcMap was used in sight distance computation, display and analysis of results whilst ArcScene was utilized for 3D views and check. In this chapter, the data preparation and the computational procedure to calculate sight distances is described.

Unlike standard procedures to estimate the ASD in 2D, 3D procedures require a depiction of the actual shape of the roadway and its roadsides. To that effect, a fundamental advantage of GIS-based techniques is the ability to facilitate the analysis of in-service highways, even if geometric design data are not available or reliable. This occurs, for example, when analyzing ASD on highways having undergone design modifications during their construction, upgraded alignment, highways lacking proper construction accuracy, sections comprising appreciable roadbed subsidence or settlements, etc.

As the accuracy of input data favors precise results, data are a major asset in a computational process. It is necessary to transform data into information suitable for analysis. Therefore, this chapter addresses the necessary data treatment to cope with the computation of sight distance on highways.

3.1.2. Road Sight Distance software

GIS-based software *Road Sight Distance* was conceived to facilitate the 3D study of sight distance on roads. It was successfully validated by Castro et al. (2014a). Once installed, this Add-in can be added to any toolbar as a button within ArcMap environment (from ArcGis 10.0 onwards). The application exploits the “GetLineOfSight” method (ESRI, 2010) from 3D Analyst toolbox available in ArcGis. This function retrieves the longitudinal profile of the terrain below a

line of sight that stretches between two desired points, determining whether the target point, besides all intermediate points of the terrain underneath that line of sight, is seen from the observer position (Figure 3.1). It can be observed that, in the example depicted in Figure 3.1, the “Get Line of Sight” method will determine that the target point is actually seen by the observer since the line of sight is not intercepted by the terrain surface.

Figure 3.1. 3D Sketch of line of sight retrieval in ArcScene.

In order to compute 3D sight distance along a highway section, the application launches line-of-sight beams iteratively from every station on the vehicle path towards the stations ahead, determining whether each target is seen by the driver. Once the loop from a station has been completed, the virtual driver is moved forward to the next station, where an identical loop is launched. According to the ASD definition, the algorithm checks lines of sight from the driver position as shown in Figure 3.2. At station i , the ASD is determined by the station $i+2$ (line of sight in green), which is the furthest one seen before the first line of sight is blocked (station $i+3$, with line of sight in red). Software stores the sight distance Boolean value (seen/not seen) of every line of sight.

Figure 3.2. ASD estimation through a line-of-sight loop.

3.1.2.1. Input data

Each particular highway requires two essential datasets for studying sight distance. First, a DEM is necessary to recreate the highway environment as the elevation of every line of sight is compared to the elevation of the model underneath to retrieve its Boolean value. Second, a file containing the set of points defining the vehicle path is needed. In addition, as stated in section 2.2.3.1, the height of the driver's eye and the target above the pavement surface must be set. The specifications of these inputs are detailed in the next subsections.

3.1.2.2. Output

The results, stored by the application after the computational process, are plotted on the sight distance graph. It is a chart in which stations are on the horizontal axis and sight distance, measured along the vehicle track, is on the vertical axis (Castro et al., 2014a). Also, software may retrieve the longitudinal profile between the observer and observed points at the request of the user. This feature permits the detection of the area that obstructs vision.

Due to the GIS geolocation capabilities, the outcome can be shown on the map. Mapped features facilitate the integrated analysis along with other factors. Furthermore, results may be exported in full detailed reports. The 3D visual inspection of the scene modelled is also possible in ArcScene to get a better understanding of modelling issues.

Figure 3.3 shows the GIS-based process to study sight distance on highways, including the input and output data as well as the auxiliary tools available in *Road Sight Distance*.

Figure 3.3. Flowchart of sight distance study procedure.

3.1.2.3. Digital elevation models

Civil engineering works are developed and supported on the terrain. The morphology of the terrain, which often affects substantially the project of such works, conditions the possible solutions. Moreover, those works modify themselves the terrain. It is thus necessary to reliably visualize both the solutions that are to be created and those facilities already existing, along with the environment.

The most adequate way of depicting the terrain on important projects is made through the use of elevation models. A DEM is a 3D model representing a terrain's surface and, occasionally, other landscape features through a collection of points or linear elements that shape terrain elevations. Given the (x, y) coordinates of a point therein, the model must enable the univocal determination of the elevation of such a point. These models are usually available either as raster bitmaps or as vector-based networks. Whilst the raster files consist of a regular grid representing the elevation, the vector-based networks are shaped by triangular irregular networks (TIN). These triangular structures are created through Delaunay triangulation, which connects all the points from the cloud of points in triples so that no other point lies within the circle defined by them on the horizontal projection. Additional information on the algorithms utilized for creating these structures is provided by George and Borouchaki (1998) and Puy (2009).

Current techniques provide cost-effective high resolution DEMs. The data to build DEM's are normally collected through different massive remote sensing techniques, such as photogrammetry or, more recently, LiDAR. Two different types of DEM could be available depending on the landscape features depicted. On the one hand, if the bare terrain is the only feature considered, a digital terrain model (DTM) is produced. A DTM is often required in highway engineering for drainage modeling, earth moving and other applications. On the other hand, a DSM additionally shapes relevant landscape elements such as trees, walls or buildings. These models, which constitute a valuable input for visualization applications, have not nonetheless exploited for ASD applications so far.

Compared to other software, the capabilities of ArcGIS in handling clouds of points and creating DEM of such a huge size make it suitable for these assignments. For example, whereas it takes 20 hours to CARD/1 highway design software to build up a 2-million-point TIN dataset, ArcGIS is capable of creating a surface out of 15 million vertices in 30 minutes using the same computer.

Two sources of data were utilized in this research for creating the necessary DEM: an airborne survey and a car-mounted terrestrial survey. Their characteristics are detailed in the next subsections.

3.1.2.3.1. Airborne-LiDAR elevation models

A first set of elevation models was provided by courtesy of the Spanish National Geographic Institute (*Instituto Geográfico Nacional* [IGN]). These models consist of raster files depicting a point cloud arranged on a 1-m sided orthogonal grid. They are models derived from airborne LiDAR. According to the survey specifications, the LiDAR resolution ensures a minimum density of 0.5 points per square meter. To achieve the abovementioned resolution, the elevation models are complemented with data from another model derived from photogrammetric restitution in areas where necessary (IGN, 2010). Such a high resolution permits that these digital representations provide a close fit to the ground surface and the elements on it.

The airborne sensors utilized allowed up to 4 signal returns per pulse; therefore, a single survey allowed to derive both a DTM and a DSM. Both elevation models were provided separately so that the points of the cloud had already been classified for the production of either model. While the upper returns would correspond to the vegetation cover and other features, the lower ones

would detect the soil. The scanning frequency of the airborne LiDAR sensor is 70 Hz and the minimum pulse frequency is 45 kHz, with a range of up to 3,000 m (IGN, 2010). The altimetry error, although depends on the areas (greater as the terrain gets rugged), is enclosed in most areas below 0.2 m. The flight for data acquisition was not specially programmed to survey the selected highway links, but meant to serve for any general purpose. Moreover, the clouds of points of both elevation models were preprocessed to produce the aforesaid layout and resolution. These models will henceforth be referred to as DTM01 and DSM01 respectively. Figure 3.4 sketches how the surveying flight collects information about the elements of the ground surface (terrain, buildings, trees, etc.) as the plane moves.

Figure 3.4. Sketch of airborne LiDAR survey.

Besides the DEM from the IGN, other DEM datasets provided by StereoCarto were used for the procedure development. The clouds of points are 2.5-meter regular grids and have similar precision characteristics than the ones provided by the IGN. The reason for using them is that, due to their smaller size, the procedure can be developed and tested quicker.

3.1.2.3.2. *Terrestrial-LiDAR elevation models*

The second source of DEM utilized to study sight distance on highways was produced through terrestrial survey. While the vehicle travels along the highway, the clouds of points are collected using onboard mobile mapping technology, named IP-S2 from Topcon®. These models from Mobile Mapping System (MMS) are featured by a high accuracy and definition. The point cloud of the MMS DSM differs from the airborne models described above since it was obtained through a mobile surveying system mounted on a car. The IPS2-Compact equipment from Topcon comprises an odometer, an IMU, a GNSS antenna and receiver, IP-S2 box, 3 class-1 scanning lasers and a 2-Mpx 360° digital camera (Topcon, 2010). According to the survey specifications, this equipment is able to capture 3D points from the roadway and the elements along both roadsides regardless of lighting conditions. The laser scanners are mounted on the surveying car so that they span the roadway as well as most elements by the roadsides: two laser devices are mounted pointing sideward from the vehicle and a third laser pointing downward at the rear of the vehicle. The typical measurement precision of these devices is ± 35

mm; their data update/output rate is 75 Hz; and their typical range is 30 m. The remaining components of the MMS equipment are used for geolocating the surveyed area. While the GNSS device sets the geospatial position, the IMU device records the orientation and the odometer measures the distance travelled and the speed. The vehicle speed during surveying was approximately 50 km/h capturing points arranged in 0.15-meter spaced profiles along the highway centerline. Crosswise to the highway centerline, the points are arranged so that they are closer and evenly spaced right below the scanner, whereas the network is more irregular and dispersed as their distance to the sensor increases.

To compare both data sources, Figure 3.5 shows the cloud of point layout. The points from the aerial DEMs (Figure 3.5a) follow a regular grid pattern, spaced 1 m apart in the case of DTM01 and DSM01, and 2.5 m apart in StereoCarto models. The layout is not aligned with the road centerline whatsoever as the surveying flight was conducted for a general purpose. In contrast, the MMS survey points are arranged in a fairly regular layout across the road centerline where the longitudinal gap is almost constant (approx. 0.15 m) and the cross distance is variable (Figure 3.5b).

Figure 3.5. Sketch of the point cloud layout obtained from a) airborne LiDAR and b) MMS.

It must be noted that whereas airborne LiDAR elevation models were available for all the highways that will be studied, MMS data only covered a few of the selected highways. The details on the models available are found in section 3.3.

Unlike in the elevation models from IGN, the data in the clouds of points obtained through the MMS survey contain the original points. However, both had to be processed. The processing is described in the next subsection.

3.1.3. Creating a cartographic database

The goal of creating a cartographic database is to bring together the necessary data in an appropriate format, taking the maximum advantage of them to perform the analysis of sight distance. Up-to-date and reliable data are necessary to analyze the current state of the highway. The capabilities of GIS's to building a cartographic database from the inputs described in the previous section is one more reason for using it.

3.1.3.1. Determination of the relevant terrain area

The first step towards a suitable cartographic database is the comprehensive characterization of the highways that are to be studied. Concretely, the horizontal projection of the highway alignment must first be defined. As-built plans of the highway are often either unavailable or unreliable. The definition of the centerline has to be nevertheless as accurate as possible, in order to determine precisely the trajectories over which the observer travels, namely the vehicle paths.

As occurred for the DEM's, the IGN provides open access to high-resolution orthophotos of the whole national territory. Such information is available online on the website (IGN, 2015). The pixel size of these orthophotos is as detailed as a 25 x 25 cm square, enabling, in principle, a good recognition of the pavement markings. Subsequently, a polyline can be accurately sketched on GIS on the centerline highway marking (Figure 3.6). For this task, a new *Feature Class* must be created in a *Geodatabase* on ArcMap.

Figure 3.6. Centerline drawn on an orthophoto from the IGN.

In the determination of the highway centerline, the continuity of the section is kept at singular spots such as intersections with traffic canalizing features or even at roundabouts in order not to atomize the sections. However, these spots shall not be analyzed in detail since the research focuses on linear sections.

Drawing centerlines is a straightforward process along most of the section, although this process is not without difficulties at few sporadic spots. Sometimes centerlines are not clearly evident because the contrast between pavement markings and the pavement surface is low, the markings are partially concealed under overhanging branches or hardly discernible due to the shade projected by features by the roadside, generally trees. Such an inconvenience was already

pointed out by Hanusch (2010). Therefore, the determination of the centerline must be checked against the azimuth graph, in order to follow theoretical alignments. The azimuth graph provides guidance on the horizontal alignments, giving a preliminary idea on the existing segments. The azimuth graph is a figure that depicts the heading direction of the highway centerline along the stations. Figure 3.7 shows such a graph, where every station on the section is represented along with its inbound and outbound heading angles. In order to ensure precision, the deflection angle between successive points on the polyline must not exceed 5 gon. Particularly on low-deflection-angle curves, this value must not exceed 2 gon. The curves are actually recreated by lobster-back segment sequences, therefore little turns produce a polyline well confined to the real centerline in position and length. An additional check is needed to ensure that all those turns follow the direction of the curve that integrates them, as occurs in Figure 3.7. If the requisites are not accomplished, the polyline sketch is refined.

Figure 3.7. Azimuth graph from the polyline sketch on highway M-600.

Once the centerline is defined, the area of terrain involved in the ASD estimation must be determined. The smaller the terrain area to be contemplated, the lesser computational time will be necessary for the sight distance calculation. Since a maximum distance of 1000 m is considered, it is advisable to create a buffer on either side of the centerline. The line input features will require a lateral distance long enough to preclude the line of sight from traversing void areas. In the worst case, a circular arc of length 1,000 m that spans a deflection angle of 200 gon would require a line of sight of 636.62 m. Considering in addition that vehicle paths are at a certain offset outwards the circumference and taking into account that roundabouts and intersections with directional islands are regarded to determine the paths, a distance of 500 m on either side was finally considered (Figure 3.8).

On the contrary, the TIN derived from MMS DSM do not cover such a large area. The whole point cloud was used to build up the TIN surface, bearing in mind that some lines of sight may traverse areas without information.

Figure 3.8. Determination of the relevant buffer area to build the DTM.

As for the orthophotos, raster terrain models were provided structured according to the sheets of the Spanish National Topographic Map (*Mapa Topográfico Nacional* [MTN]). Each sheet is divided into 64 rectangular cells which correspond to an ASCII raster file each. The buffer shape determines which cells will be used to build up the terrain model (Figure 3.9).

Figure 3.9. Selection of the DTM cells overlapped with buffer.

It is noticeable that void areas appear in Figure 3.9 in the elevation model raster file. These tiles correspond to large surfaces such as reservoirs, where bathymetry data could not be collected by aerial LiDAR. In addition, tiles on built up parcels and beneath bridges lack information in the DTMs. However, neither of them interferes in the study of sight distance. For obvious reasons, the elevation data of reservoirs are not relevant. The elevation data built-up parcels can be well

interpolated whilst the area beneath a bridge can be covered up with the data of the corresponding DSM. The steps to overcome this problem are explained in the following pages.

The next step is the conversion of those raster files to point shapes. After that, the selection of points laying within the 500-m buffer is necessary. The tool *Intersect* of the ArcMap ArcToolBox does that procedure. This way, the number of points considered for the final DTM is drastically minimized, hence the computational time is highly reduced. ArcGis enables the automation and management of these workflows by means of Model builder. The automated process workflow is exhibited in Figure 3.10, comprising the creation of the buffer, the necessary format conversions of the elevation models and their intersection.

Figure 3.10. Model builder workflow.

The initial centerline is used for three more purposes besides those explained in the previous subsection. It will be utilized to deduce the vehicle paths, to define the roadway area and to extract the theoretical alignments of the highway. The two first steps will be explained in the following subsections whereas the alignment data extraction process will be explained in section 3.2.

3.1.3.2. Obtaining the vehicle path

The centerline is first doubled creating two new polyline *feature classes*. They will be edited in order to fit the trajectory features for each direction: outward path centerline and return path centerline. Such features must include roundabouts, directional islands and other features at intersections. Hence, each one follows exactly the former centerline except in sections where the two regular lanes decouple. The pavement markings on the inner lane edge aid in determining these polylines where possible. Otherwise, the imaginary equidistant boundary of the trajectory described by a vehicle merging the roundabout is represented. It must

nonetheless be kept in mind that these parts of trajectories is not relevant for the scope of this thesis as the sight distance is not studied there.

Once the two new path centerlines are defined, the *move parallel* tool of *Road Sight Distance* is used to derive the vehicle paths from either centerline polyline. The due displacement value is measured from the centerline to the right side so that the path location complies with the criteria of the former Spanish standard for geometric design (Ministerio de Fomento, 2000), which established such path on the left of the outer lane edge at an offset of 1.5 m. This criterion was selected rather than the current one as it is considered more in line with real vehicle trajectories. The criterion of the current standard (Ministerio de Fomento, 2016), which sets the trajectory at an offset of 1.5 m from the centerline, yields unrealistic path locations on narrow cross sections (see section 3.3 for the cross-section features of selected highways).

Then the resulting polyline is discretized into a set of stations using *calculate stations* as the sight distance computation requires a set of stations. The spacing distance between consecutive points can be selected by the user. It is worth noting that software requires either a *point feature class* in a *Geodatabase* or a point shapefile to be launched on. Although in this research, these inputs came from the centerline sketch, an alternative procedure could be the use of a set of stations derived from one or several GPS tracks. The advantage of using field data to recreate the vehicle path is that realistic trajectories are considered instead of the theoretical ones recommended by the applicable standard.

Figure 3.11 provides a graphical example of the feature classes created from the highway centerline (green line). It also plots both filiatiions of the centerline for each travelling direction, which decouple at directional islands to enable a reliable representation of the vehicle paths, and the corresponding stations as red and blue dots.

Figure 3.11. Centerline and path feature classes at an intersection.

3.1.3.3. Processing the elevation model data in the roadway zone

The use of elevation models derived from massive remote sensing techniques, either airborne or car-mounted equipment, has some inherent limitations. In both types of models, elevation data usually contain cumbersome spots for estimating sight distance.

First, items not forming part of the static landscape are captured during the survey, such as vehicles or even wildlife. Although these elements can presumably only be found in DSMs, the preprocessing algorithms utilized to deliver the DTM might not have treated such spots adequately. Therefore it is necessary to ensure that the modelled roadway area is clear from such unrealistic hindrances.

In a surface built up from a point cloud by means of a Delaunay triangulation, a univocal correspondence must exist between the coordinates (x, y) on the horizontal projection of every point on the defined surface and its elevation. In other words, points on the same surface cannot share identical (x, y) coordinates and have, in contrast, different elevation values. The intrinsic definition of a Delaunay triangulation impedes the overlapping of elements in the plane, i.e. the triangles created are arranged in such a way that they can only connect points whose distance in the plane is minimum, even if there are closer points from a 3-D approach. Since no overhanging features are allowed, it is difficult to represent these frequent structures by the roadsides such as overpasses, tree branches, streetlights, gantries, cantilever signs, aerial power lines, etc. Delaunay triangulation connects points in the overhanging element with points on the pavement surface, creating a sort of wall that does not actually exist. The free space below such elements is indeed obstructed by the theoretical surface of the DSM (Figure 3.12) thus the representation of the reality is distorted and not suitable to enable a lifelike ASD estimation. Hence all these points are entitled to be removed or replaced depending of the type of model.

The shape of roadside elements is another fact to bear in mind. DSMs from airborne LiDAR can hardly model small vertical roadside features, such as traffic signs or guardrails, although they could represent considerably larger vertical devices (e.g. the cantilever sign in Figure 3.12) depending on the resolution. In general, roadside equipment is better depicted by DMSs derived from terrestrial LiDAR.

Figure 3.12. a) Cantilever sign, and b) representation of such a sign by a DSM derived from airborne LiDAR.

Furthermore, as aforementioned, the pulse returns in the raw airborne LiDAR data underwent processing to identify whether they make up the DTM or the DSM. For example, bridges and overpasses are contemplated as landscape features other than the bare terrain. Therefore these features are considered, in principle, by the DSM only and not by the DTM. Nonetheless, it is fundamental that the roadway is included in such models, regardless of it is settled on a fill section, a cut section a bridge or a tunnel.

The issues above described were observed at the pilot stage. In the light of this, the processing tasks that will be described in the following were devised. To prevent distortion in the analysis of sight distance, the different DEM were processed in diverse ways depending on the features of the model to obtain a lifelike and clear roadway recreation.

In the case of DEM from airborne LiDAR survey, since a DTM and a DSM are available, points of both models can be combined to overcome the difficulties arisen in the roadway area at the spots above described. Therefore, the points from the DTM in the roadway zone will be utilized to recreate the most of the roadway in both models whereas the points from the DSM shall be used on bridges and overpasses if any. Figure 3.13 shows a scheme of the selection of points in the cloud of points that are replaced by those of the DTM on the roadway.

Figure 3.13. Scheme of overhanging elements removal.

On the basis of the vehicle paths, the roadway zone can be determined. It covers the paved area, therefore, paved directional islands must be embedded at intersections. A *buffer* feature class must be created around each vehicle path according to the cross section dimensions. If any void area corresponding to a paved area remains between both buffers, another polygon shape must be created to cover it so when they all are combined with the geoprocessing tool *merge*, the resulting area matches the paved zone. This well-defined area is used to determine the points of the DTM cloud within the roadway area and create a new feature class through the tool *intersect*. Since there is an overlapping area between adjacent ASCII raster files (depicted in Figure 3.9), the duplicated points coinciding in that area must be removed. It is recommended that they all be merged into a new feature class afterwards. Then the next step is to *create TIN* using the roadway points for its visual inspection. This step facilitates the detection of cumbersome spots and diagnose if DSM points are needed in order to build up any bridge or overpass in the elevation model.

3.1.3.3.1. Elevation smoothing algorithm

Also in the pilot phase, the necessity of a roadway smoothing process to minimize elevation error in the roadway zone manifested. The roadway is the most critical zone of the terrain model because the height of the observer is based on the model surface and, more often than not, the

lines of sight sway over it, hence the highest elevation accuracy is needed. In addition, when sight distance is calculated through design software, the theoretical pavement surface is utilized whereas the rest of the area is depicted according to a terrain model.

The smoothing algorithm was programmed in MATLAB. For each point, two vicinity areas are defined (blue point in Figure 3.14): the closer vicinity area are points that lay closer than 1.5 m, which correspond to the green points in the Figure 3.14, provided that the grid is spaced 1 m apart. The further vicinity area comprises points laying at a distance between 1.5 and 2.3 m from the considered point (orange points in the Figure 3.14 in a 1-m spaced grid). It first determines the points of central area of the roadway, which corresponds to points whose closer and further vicinities lay within the roadway zone. Those points will be useful in the filtering criteria. Next, the algorithm performs an outlier detection, i.e. points whose elevation differs much from the average elevation of the surrounding points. If the difference in absolute value between the elevation of the point and the elevation of the closer point in the central area is high, the elevation of the point is recalculated as a function of the closer central point height and the average values of both vicinity areas. This step was meant not only to amend random erroneous elevations but also to correct the elevation of depressed areas near ditches and at cross-drainage culvert ends, where the DTM points occasionally represent their base height in the roadway zone. The elevation smoothing part itself is computed in the third step of the routine, where the elevation of every point is recalculated as the mean elevation of the points in the nearer vicinity area and the point considered itself.

Figure 3.14. Point of the roadway area and its vicinity zones.

Figure 3.15 illustrates the hybridization process of the elevation model where a singular spot is found. In this case a noticeable terrain discontinuity in the route is evident in Figure 3.15a. The garnet broken line in Figure 3.15a plots the centerline profile of the original terrain model where the dell can be observed. Also, the lack of elevation data manifests in the overpass area as larger triangles in the TIN depiction to cover the void areas. Figure 3.15c depicts the elevation model points selected to recreate the roadway zone at and near the overpass. The TIN shaded relief facilitates the identification of the DEM points that cannot display correctly the roadway. Finally, Figure 3.15 exhibits the final appearance of the terrain model after the overpass is built up and the roadway zone is smoothed, where the roughness reduction is evident.

Figure 3.15. Recreation of a bridge in the elevation model a) original DTM TIN; b) centerline profile before and after; c) roadway points considered for recreation, and d) processed DTM TIN.

3.1.3.4. Processing the terrestrial LiDAR elevation model

In the case of the MMS DEM, the density of points of the model in the roadway area reaches as much as 200 points per m^2 . This makes it possible to dispense with points captured from elements other than those of pavement, i.e. overhanging elements, vehicles, aerial power lines or wildlife. The points of the roadway area account, moreover, for more than half of the cloud of points. Ever after removing those non-relevant points, the number of points is still in overabundance. So many points are excessive, repetitious and reduce the computational performance of sight distance calculation without increasing precision in results.

A classification algorithm was used in ArcMap to detect points not corresponding to the roadway. First, the TIN surface was created with the raw cloud of points. Next, the centerline polyline was given elevation by means of shape interpolation on this TIN surface. Random decimation of points is carried out, preserving 25 % of the initial points in the roadway. Finally, the elevation of the remaining points within the roadway was compared to the elevation of the nearest centerline point. Points were removed if that difference exceeded 0.2 m. In this respect, the roadside borders may contain points of metallic barriers that were not removed.

Figure 3.16 represents a plane view of the MMS DSM TIN surfaces before and after being processed. A tree, whose crown partially overhangs the roadway, is visible between stations 4+585 and 4+590 (westbound) in Figure 3.16a. Such an obstruction would block vision

completely. In the corresponding view shown in Figure 3.16b not only the overhanging points were removed but also the TIN contains much less nodes in the roadway area, the model providing a more lifelike display of the visibility conditions.

Figure 3.16. a) MMS DSM with a tree overhanging the roadway, and b) the same elevation model after being processed.

3.2. Alignment data extraction

This part of the study plays an important role as alignment features will be analyzed in this research. Alignment data are a necessary asset in highway inventories. Frequently, these data are, however, not available and if otherwise, they are not accurate or not reliable. This occurs often owing to the fact that construction works might have not been completed in full accordance with the corresponding project specifications. Also, alignment upgrade, restoration, or any other sort of geometry alteration might have been performed during the service life of facilities.

As stated in the literature review, the problem of alignment restoration can be tackled as an optimization problem. To find the solution, it turns out, nonetheless, that the problem cannot be stated analytically since the objective function has to be defined piecewise in such a way that even the sub-domains are delimited by variables. Therefore, it requires some kind of numeric algorithms to be solved.

In accordance with the literature review, azimuth graphs and slope graphs deliver valuable information for the alignment recreation, especially for segmentation and alignment parameter deduction. However, it should be accompanied by planimetric and altimetric adjustment respectively. Curvature-based fitting methods were dismissed as the datasets are highly affected by noise perturbation and would produce dramatically biased results. Moreover, they are virtually useless to detect low deflection angle curves and large radii curves. This section describes the alignment deduction process carried out and the elements utilized for that purpose.

3.2.1. Horizontal alignment data extraction

The horizontal alignment consists of a sequence of three different types of elements. Tangents, whose curvature is null, support the geometric structure of the horizontal alignment and are connected by curves. Circular arcs, whose curvature is constant, can be connected either to

tangents or to other curves. Spirals, generally in the shape of an Euler spiral or clothoid, provide smooth transition between segments which have different curvature by means of a linear variation of the curvature.

As aforementioned, a precise definition of the centerline polyline ensures a more trustworthy alignment result. The centerline polyline, sketched as described in section 3.1.3, is intended to be an implied and imaginary line depicting the central axis of the roadway that will enable the adjustment of a sequence of alignments. It is recalled that although directional islands (i.e. at intersections) were considered for vehicle path, they were disregarded in alignment recreation as these parts will not be analyzed as intersections themselves.

The polyline vertices define a sequence of (x_i, y_i) coordinates. They serve as basis to deduce the stationing according to the equation that follows:

$$\Delta s_i = \sqrt{(x_{i+1} - x_i)^2 + (y_{i+1} - y_i)^2} \quad (3.1)$$

Where coordinates (x_i, y_i) represent point i located at distance s_i from the beginning of the alignment. The azimuth is calculated as follows:

$$\theta_i = \begin{cases} \arctan\left(\frac{y_{i+1} - y_i}{x_{i+1} - x_i}\right) & \text{if } x_i \neq x_{i+1} \\ \pm \frac{\pi}{2} & \text{if } x_i = x_{i+1} \end{cases} \quad (3.2)$$

It is worth noting that the theoretical alignments must first fit into the sequence of vertices of the polyline and fulfill the boundary mathematical conditions of the geometric elements that make it up. These criteria prevailed over the adjustment into the curvature or the azimuth datasets.

The polyline points are exported from ArcMap to AutoCAD and then to MS Excel. Alignment segmentation is facilitated by examining the azimuth graph whereas the adjustment of alignments is made in CAD software. Figure 3.17 depicts the same azimuth dataset as in Figure 3.7, yet the theoretical azimuths of the deduced alignments are also shown. Horizontal segments such as those located between stations 0+000 to 0+370 and 2+240 to 2+490 indicate that a tangent is located there as on these elements the azimuth remains constant. According to the standard azimuth measurement, a circular arc to the right exists where the line follows a rather straight upward trend, i.e. constant slope, whereas the analogue downward line denotes a circular arc towards the left. The tangent-to-curve transitions can be rendered by means of spirals if there is a set of azimuths following a parabolic trend that connects both segments smoothly (for example, stations 1+030 to 1+130 approximately) or can be direct if no such a changeover is observed (station 2+485). Similarly, curve-to-curve transition may exist on compound curves, whether or not a spiral connects them. Reverse curves, connected by means of two spirals stitching at their null-curvature ends, are also frequent (stations 0+660, 1+400 and 1+800).

Figure 3.17. Azimuth graph of the polyline and the deduced alignments on highway M-600.

The curve radius values were derived through linear regression on the azimuth sets that showed a fairly constant slope. The regression coefficient multiplying stationing is the curvature value as:

$$k = \frac{\Delta \theta}{\Delta s} \quad (3.3)$$

θ being measured in radians. Therefore the deduced radius is given by:

$$R = \frac{1}{k} \quad (3.4)$$

The sequence of alignments is completed by the insertion of the due transition curves. To define these elements, automated computation of the spirals was programmed in VBA in MS Excel. It calculates numerically the spiral length, the deflection angle α_L and the location parameter x_0 with 10 decimal places precision from the circular curve radius R and the curve setback ΔR (Figure 3.18).

The spiral location parameters ΔR , x_0 and α_L are given by the equations that follow (Kraemer et al., 1994).

$$\Delta R = \frac{L}{2} \sum_{n=0}^{\infty} (-1)^n \frac{\alpha_L^{2n+1}}{(4n+3)(2n+2)!} \quad (3.5)$$

$$x_0 = \frac{L}{2} \sum_{n=0}^{\infty} (-1)^n \frac{\alpha_L^{2n}}{(4n+1)(2n+1)!} \quad (3.6)$$

$$\alpha_L = \frac{100 \cdot L}{\pi \cdot R} [\text{gon}] \quad (3.7)$$

Figure 3.18. Spiral geometric parameters.

The first ten terms of the series are used in calculation. As stated above, L , α_L and x_0 are the unknowns. The VBA script utilizes the Newton-Raphson algorithm to solve the equation system. The modified Newton-Raphson method for equation systems can be expressed as follows (Ben-Israel, 1966):

$$\mathbf{x}_{p+1} = \mathbf{x}_p - J^{-1}(\mathbf{x}_0) \mathbf{f}(\mathbf{x}_p) \quad (p=0,1,\dots) \quad (3.8)$$

where the vector $\mathbf{x}$ contains the cited unknowns; J is the Jacobian matrix of expressions (3.5), (3.6) and (3.7) equaled zero; and the number of iteration steps is p . After a number of iterations, the equation (3.8) converges to a solution of (3.9):

$$J(\mathbf{x}_0) \mathbf{f}(\mathbf{x}) = 0 \quad (3.9)$$

The following borderline cases found in the alignment recreation of the selected highways are worth noting:

- Vertex spirals, namely, curves in which the inbound and outbound spirals are connected by a null-length circular arc. This occurred where, for the radius deduced value, the adjustment of both spiral yielded curves exceeding the bisector each. Then the vertex spirals were assumed to be symmetric and the bisector distance was the initial parameter used to deduce the curve radius.

- Multiple compound curves (in some cases more than 3) were sketched where spirals could not be adjusted satisfactorily between a tangent and a circular arc or between two circular arcs.
- Very-low deflection angle curves (less than 1 gon), detected through the precise elaboration of the azimuth graph.

3.2.2. Vertical alignment data extraction

The vertical alignment of highways consists of a sequence of grades and vertical curves. The grades are profile elements where the angle of inclination with respect to a horizontal line is constant. On vertical curves, the grade varies linearly over distance, which geometrically yields a parabola.

In the view of the diversity of criteria across different design guides and standards, it is worth mentioning the parametrization of vertical curves in this thesis is expressed in accordance with the Spanish standard (Ministerio de Fomento, 2016). Here, the vertical curve parameter K_V stands for vertical curvature, i.e. the radius of the osculating circle of the parabola at its vertex, expressed in meters. The value of K_V is negative when the curve is a crest and positive for sag curves.

The polyline *Feature Class* that defined centerline on the orthophoto is used to define the most likely sequence of profile alignments. To this end, the terrain profile is unfolded spatially under the polyline in order to obtain the highway longitudinal profile. First, a new point-type *Feature Class* is created out of the centerline polyline using the tool *Create stations from polyline* available in the Add-In *Road Sight Distance*. The spacing is set 1 m apart to obtain the profile with enough accuracy. Next, the *Interpolate shape* tool available in the 3D Analyst toolbox is launched to provide the points created with the elevation of the processed TIN. These elevation values will define the vertical profile of the highway; thus, the elevation coordinates are ellipsoidal. Next, the attribute table, where the geometry of points has already been computed on UTM coordinates is exported as a *.dbf file to be handled as a spreadsheet. Next, the points are imported in AutoCAD, where the vertical dimension is scaled 10:1 to better appreciate the alignments in profile. The sketch of the horizontal alignment was carried out aided by AutoCAD®, versions 2012 and 2014. The grades are first identified where the points clearly follow a linear trend and determined through linear regression. The required crests and sags are placed between successive grades. Crests and sags are parabolas tangent at the stitching points with other vertical alignments. The intersection of the adjacent grades enables the determination of the parabola parameter through the vertical gap value f at that point (elevation difference between this intersection point and the parabola right below):

$$f = \frac{L}{8 \cdot K_V} \Rightarrow K_V = \frac{8 \cdot f}{L^2} \quad (3.10)$$

Figure 3.19 exhibits an example of vertical alignment fitting into the terrain profile. The defined alignments fit approximately into the interpolated terrain elevation values.

Figure 3.19. Example of profile reconstruction on highway M-104.

When the expression in equation (3.10) produces a partial superposition of two vertical curves in the first estimation, the existence of a reverse curve on the vertical alignment is assumed. The alignments involved have to be recalculated so that the crest and grade are directly stitched. A Newton-Raphson algorithm was programmed in Excel VBA in order to resolve the resulting non-linear equation system. The grades before the first curve and after the second curve maintain their geometry except for the stitching points with the new curves. The interim grade (whose length must be null) turns around its middle point (s_m, z_m). This point is not necessarily the stitching point between the crest and the sag curve since the reverse curve is generally non-symmetrical. The unknowns of the equation system are the grade between the crest and sag station of the intersection g_1 , the station where first and second grade would intersect s_1 , the elevation of such and intersection z_1 , the station where second and third station would intersect s_2 , and the elevation of the latter one z_2 as shown in Figure 3.20. The solution of the equation system yields the blue grade in Figure 3.20, which is tangent to both the crest and the sag at their stitching point.

3.3. Selected highways

3.3.1. Introduction

In order to test the methodology hereby proposed, several in-service highway links were selected. In this section, the main geometric features of these selected highways links are described. On the one hand, a set of two-lane rural highways were chosen. This set will be subject of the sight distance, 3D alignment coordination and traffic safety analyses. On the other hand, a German motorway section was selected to carry out the study of spatial alignment coordination.

Figure 3.20. Example of vertical reverse curves adjustment.

3.3.2. Geometric characterization of the selected two-lane highways

A set of rural highways totaling 122.7 km has been selected for the analysis. All of them are two-lane paved highways, with no climbing lanes whatsoever. These sections are part of the regional highway network of Madrid. They were selected through a visual inspection on *Google Street View*[®] according to two main criteria. First, they must show potential sight distance and alignment coordination issues. Second, they must cover a broad range of design speeds. Some of them belong to the main network (M in column “Network” of the Table 3.1), some others to the secondary (S) and the rest are part of the local network (L). The Spanish standard (Ministerio de Fomento, 2000), which is the subsidiary norm for all the highway networks, set out 4 design speed values²: 100, 80, 60 and 40 km/h. The design speeds were assigned to sections as the most likely ones according to their geometric features, knowing beforehand that all technical specifications and requirements may not be met. Actually, as it was already stated, one of the motivations of this research is to provide insight of the effect of such deviations from standards. The selected highways cover all the standardized design speeds in the abovementioned instruction.

The general criteria used to assign a design speed to the highway links is based on the analysis of the horizontal and vertical alignments. Standardized horizontal curves are associated with a characteristic speed and so does vertical curves and grades. To link a design speed with a highway section, no more than 10% to 15% of the curved length (circular arcs) must not comply

² As the current Spanish geometric design standard (Ministerio de Fomento, 2016), which for the first time does contemplate more design speed values, came into force recently, it is unlikely that the assumed design speeds were set at values other than those presented.

with the corresponding standardized value. An identical criterion was used in alignment profile features, but it must be borne in mind that vertical curve parameters below standard do not necessarily yield sight distance issues where the segment length is shorter than the ASD. Even in some of the selected links, the design parameters are not consistent between the plan and the profile in terms of those characteristic speeds. In addition, two of the selected links are far from meeting the lowest standard ($v_d = 40$ km/h), herein after referred to as off-standard. The results of this classification are shown in Table 3.1. Detailed information can be found in the Appendix I of this thesis.

Table 3.1. Main features of selected two-lane rural highway links.

Highway	Section length [km]	Terrain type	Design speed [km/h]	Traveled-way width [m]	Network
M-104	10.952	Rolling	60	6.5/7.5	S
M-221 / M-222	13.265	Rolling	80	6/7	S/L
M-325	15.053	Rolling	< 40	5.5/5.5	L
M-513	9.056	Rolling	40	7/9	S
M-600	16.782	Hilly	60	7/9	M
M-601	9.055	Rolling	60	7/9	M
	1.358	Hilly	40	6.5/7.5	M
M-607	12.284	Rolling	100	7/12	M
M-610	8.291	Rolling	40	6.5/7.5	S
M-611	5.072	Rolling	40	6.5/7.5	S
M-629	15.266	Mountainous	< 40	5/5	L
M-633	6.214	Rolling	40	5.5/6	L

Figure 3.21 shows the location of the selected highway links in the region of Madrid. Most of them are found in the north or northwestern area of the region, whereas two of them are located in the southeast.

Figure 3.22 provides an impression of the highway M-607 from the point of view of a driver. Its design speed is presumed as 100 km/h. The generous alignments and its wide shoulders can be observed. Also from the driver's perspective, a depiction of the view if highway M-222 is shown in Figure 3.23. The design speed was 80 km/h. In this case, the cross section is slightly narrower than in the previous figure and the alignments still somewhat smooth, but appear sharper than in the previous case. Finally, Figure 3.24 shows the Canencia mountain pass (highway M-629), also from the point of view of a driver. According to the assessment shown in Appendix I, its geometric design is far from meeting the standards for the lowest design speed value (40 km/h). The winding alignments are noticeable along with the narrow roadway and clearances.

With regard to the statistical analysis of safety performance, no changes in the geometric design were produced in any of the selected highways over the period analyzed.

3.3.2.1. Available surveying data

The available surveying data of the selected highway links for the elaboration of this thesis comprises airborne and terrestrial DEM (Table 3.2).

Figure 3.21. Location of the selected highway links.

Figure 3.22. View of highway M-607 from a driver's perspective.

Figure 3.23. View of highway M-222 from a driver's perspective.

Figure 3.24. View of highway M-629 from the driver's perspective.

Table 3.2. Summary of the surveying data of the selected highways.

Highway	Airborne LiDAR	Terrestrial LiDAR
M-104	IGN DTM01 DSM01	-
M-221 / M-222	IGN DTM01 DSM01	IP-S2
M-325	IGN DTM01 DSM01	IP-S2
M-513	IGN DTM01 DSM01	IP-S2
M-600	IGN DTM01 DSM01	-
M-601	IGN DTM01 DSM01	IP-S2
M-607	IGN DTM01 DSM01 StereoCarto2.5	IP-S2
M-610	IGN DTM01 DSM01	IP-S2
M-611	IGN DTM01 DSM01	IP-S2
M-629	IGN DTM01 DSM01	IP-S2
M-633	IGN DTM01 DSM01	IP-S2

3.3.2.2. Remarks on the alignment recreation of the selected highways

The procedure for the recreation of alignment detailed in section 3.2 aimed to determine the most likely horizontal and vertical alignments. The alignment data extraction issue is a problem highly exposed to either bias or variance depending on the technique used. Thus, it is difficult to obtain an accurate outcome. In general, highways designed according to higher standards resulted easier to derive their alignments for several reasons:

- The elements that make up alignment are generally longer and thus their parameters can be inferred with higher accuracy, except for large-radius curves if the data extraction procedure is based on curvature analysis.
- Low-standard and off-standard highways contain shorter alignments and more complex successions of elements since, besides their lower design speeds, the geometrics must be tailored to more strict conditions (topography, right-of-way restrictions, etc.) adopting less clear layouts.

- The construction of medium and high-standard highways is submitted to a more exhaustive control, particularly concerning the geometrics, which enables the correct recognition of alignments and an unbiased parameter estimation in recreation. Almost regardless of the recreation technique (classic topography, polyline on orthophoto, surveying vehicle, etc.), the increased precision of road markings when executed will be a relevant factor for the results.
- Some of the highway links analyzed in this thesis, especially those categorized as off-standard, might have not been designed in accordance with modern standards. Only if they have been upgraded, they might have adopted featured standard alignments, yet they probably did not if they have undergone resurfacing, restoration or rehabilitation (RRR) only. According to the most extended practices when upgrading the alignment, much more attention is paid to the horizontal projection than to the profile. This may produce discontinuities on the vertical alignment that hardly fit into a theoretical profile.

3.3.3. Geometric characterization of the selected motorway section

A case study from Federal German Motorways has been considered to research the assessment of 3D alignment coordination of motorways. It was selected according to the data availability. A section comprising 103.5 km of the German federal motorway BAB9 was chosen to study the alignment coordination and the development of standard spatial elements for motorways. It is located on a rolling terrain, where spatial deficits in the perspective are more likely to occur. The section selected comprises part of the route that drives from Berlin to Munich, in northern Bavaria. This section is part of one of the oldest motorway routes in Germany, designed and constructed before the 1940's. The geometric design has undergone a major upgrade, completed a decade ago. The alignment data were extracted from the track of a surveying vehicle equipped with GPS, travelling on the motorway each direction twice. From those tracks, the geometrics were built up in software CARD/1. The horizontal alignment is composed of 106 segments and the vertical alignment comprises 229 elements. Regarding the cross section, it was widened from four lanes without emergency lane to six lanes plus emergency lane during the upgrade. Figure 3.25 displays the appearance of the motorway today.

Figure 3.25. View of motorway BAB-9 from a driver's perspective (Panoramio, 2015).

Chapter 4

THREE-DIMENSIONAL SIGHT DISTANCE MODELLING

4.1. Introduction

In this chapter, the results of the 3D sight distance computation performed in *Road Sight Distance* are presented and discussed. The computation was made using three different types of DEM, they being derived as described in section 3.1. Then, the sight distance results obtained with the different elevation models are analyzed and compared through several examples. In the sight distance graph, it is possible to visualize the unlike outcomes, identifying the sight distance reduction caused by obstructions. Furthermore, a preliminary comparison between the corresponding ASD values and those required by the standard (SSD) is made.

4.2. Methodology

Road Sight Distance software was utilized to compute sight distance along the selected highways. The inputs and the details to elaborate a suitable cartographic database for the study of sight distance using such an application were described in section 3.1.3. As stated there, the elevation models available for sight distance modelling in this research were DTM01, DSM01, and MMS DSM.

The geometric parameters that characterize the lines of sight were taken in accordance with the former Spanish geometric design standard (Ministerio de Fomento, 2000), in order to be suitable to study the compliance of required SSD. The height of the driver's eye above the pavement surface was taken to be 1.1 m, which corresponds to the view from a passenger car. The height of the target object above the roadway was taken to be 0.2 m. In the current standard (Ministerio de Fomento, 2016), the target height rises up to 0.5 m. Such a changeover does not seem to respond to specific alignment design criteria, but was rather discretionary selected. For this reason, the criteria of the former standard are taken. In addition, as the altimetry error of the elevation models are enclosed below 0.2 m, this value appears sound.

Both the observer and the target move over stations located on a line parallel to the centerline at an offset of 1.5 m from the outer lane edge, also in accordance with the former standard (Ministerio de Fomento, 2000) for the reasons explained in section 3.1.3.2. These stations were set spaced 5 m apart.

4.3. Results and discussion

The sight distance output is displayed in the sight distance graph in *Road Sight Distance*. The interpretation of such a graph is paramount for the analysis. In this respect, during the pilot study, the reliability of results produced by *Road Sight Distance* software was checked. In the MMS survey, 360° images of the environment were taken along the path travelled by the surveying vehicle. Such snapshots also contain information of the position where they were taken. A detailed visual inspection was also performed on the spot, and the whole highway section was filmed from the moving vehicle. Therefore, the real sight distance conditions by the roadside could be checked in all cases, and particularly those cases where the sight distance outcome could not seem realistic.

The features of the different elevation models available provide unlike representations of the road landscape. Thus the use of an elevation model over other models is by no means irrelevant here. The elevation models used reveal the differences produced in the sight distance outcome as the effect of the landscape elements considered by one of the models compared and not by the other one. In this section, this effect will be analyzed from the point of view of sight distance outcome and, second, from the standpoint of the resulting ASD.

4.3.1. Effect of roadside elements on sight distance results

By examining the sight distance graphs, the differences in results can be seen straightforwardly. The impact of vegetation on sight distance results is present over most of the selected highways. A section of highway M-607 near a mound, which is covered with trees and bushes, was chosen to illustrate the effect of vegetation. Figure 4.1 shows the real scene from the driver standpoint, where both the mound and the roadway section seen beyond it are pointed. As no overhanging item is on the roadway, a DSM performs a more accurate representation of the scene than the corresponding DTM. According to the study using the DTM01, the mound causes a visual interruption of circa 40 m long that ranges 720 m. This can be observed in Figure 4.2 (superimposed sight distance graphs of the DTM and the DSM results), represented by the slim red diagonal strip spanning between letters 'a' and 'b'. Nevertheless, in the study carried out using the DSM01, as vegetation is taken into account, the successive hidden sections produced are larger whilst visible section beyond is much less noticed, a fact more tied in with reality. As it can be seen in Figure 4.2, the differences in the results obtained using different DEMs are significant not only for the ASD values, but also in the length of the hidden and reemerged sections where they occur.

The effect of roadside equipment items, such as signals or barriers located by the roadsides, on sight distance is also noticeable. In general, these items are fundamentally vertical; therefore, they can hardly be detected by an airborne LiDAR survey. Except for considerably large devices, the geospatial characterization of roadside equipment is captured when surveying with terrestrial equipment only. An exception to this was shown in the literature review (cantilever

sign in Figure 3.12). That is why points above the roadway were removed while processing the elevation model.

Figure 4.1. Mound causing hidden section and visible roadway beyond in M-607.

Figure 4.2. Sight distance graph comparing the outputs using DTM01 and DSM01 in M-607.

Route confirmation signs and other large traffic signs existing by roadsides could change the sight distance outcome when the MMS DSM is used, although these items just cause small hidden sections (from 5 to 10 m long). Its effect on sight distance is more noticeable when they are located by the inner side of a right bends and/or just before a sag vertical curve. An illustrative example of this fact can be found on highway M-221, where such equipment is present. Several signalization panels are located before a crest curve, having a null footprint on the sight distance graph. On the contrary, those set at the beginning of a sag curve overlapped

with a right bend do produce that some stations ahead cannot be seen from driver's position. To this end, sight distance results utilizing the DTM01 and the MMS DSM are examined in this example. Not only do large signs have influence on visibility but also road signs such as the speed limitation sign depicted in Figure 4.3 produce the vertical row of non-seen stations while a driver is at station 440, denoted with 'a' in the graph shown in Figure 4.4. Also, the metallic barrier highlighted in Figure 4.3 causes an interruption of sight distance (slightly horizontal area in light green marked as 'b' in Figure 4.4). Items by the roadsides other than roadside equipment and plants reduce likewise sight distance. For instance, a wire fence that arises besides the roadway is observable in Figure 4.3. The MMS devices captured the points of such fence due to its high sensitivity. As a result, the Delaunay triangulation was built up in such a way that the surface of the MMS DSM depicts an opaque wall. In addition, the wire fence is placed by the inner roadside of a horizontal curve, but enabling a section of roadway being visible through it (highlighted in Figure 4.3). However, the fence concealed that section according to the sight distance output when using a MMS DSM. In Figure 4.4, the dark green area on the right of letter 'c', from station 400 on, corresponds to the station where a driver would have left the fence behind, and therefore a road section is seen beyond the ASD. Conversely, according to the sight distance calculation output using the DTM01, the mentioned roadway stretch is actually seen through the wired fence much before (area shaded in light green marked as 'd' on Figure 4.4). Furthermore, points of the overhead power line highlighted in Figure 4.3 were captured by the terrestrial survey using IPS2 equipment. Therefore, the cloud of points was processed in order to prevent the existence of an "opaque wall" blocking the road in the MMS DSM.

Figure 4.3. Items by the roadsides that reduce sight distance in M-221.

4.3.2. Comparison of available sight distance results

In this subsection, the ASD values produced using different elevation models will be statistically compared. For this purpose, the results on two of the selected highway links (M-221 and M-325) in both directions will be analyzed. Their roadsides are scattered by many sight distance

restrictions (especially the latter one) and, by just studying those two highways, statistically significant differences arose in the results.

Figure 4.4. Sight distance graph comparing outputs from a DTM and a MMS DSM in M-221.

The final bounds of each trajectory, where the ASD equaled the distance to the latter station on the path, were removed from the sample. As in the previous subsection, three available DEM were used as inputs: DTM01, DSM01, and DSM MMS.

Table 4.1 exhibits statistical values of ASD of highway M-221. The central tendency parameters (average and median) show that when a DTM is used, the resulting ASD values were greater than those of the DSMs, as it could be expected. Regarding the spread of values, the results of the DTM showed the greatest dispersion. The coefficient of variation is a measure of spread that describes the amount of variability relative to the mean. Because the coefficient of variation is unitless, it can be used instead of the standard deviation to compare the spread of data sets that have different units or different means. In this case, the spread results more alike for the three samples, around 55%. Finally, as an indicator of sight-distance related hazardous zones, the number of stations where the ASD did not comply with standard SSD is provided. The estimation of required SSD contemplated the average grade on the highway vertical profile from the station considered to 120 m ahead, as this is the rounded SSD value for design speed 80 km/h. The dissimilar values depending on the elevation model used highlight their important effects on ASD.

Table 4.2 compares statistical values related to ASD of highway M-325. As occurred for highway M-221, the resulting central tendency parameters resulted smaller when the MMS DSM was used and greater when the DTM01 was utilized. The coefficients of variation are more different than in the previous highway because the presence of elements by the roadside, especially vegetation, is more extensive in this case. The estimation of SSD accounted for the grade over the 40 m that follow the station considered (rounded value of required SSD for design speed 40 km/h). It is worth recalling that this highway link was classified as off-standard in chapter 3.3.

The shares of highway section where ASD does not meet the standard SSD resulted also somewhat different among them, but not as high as on highway M-221. This apparently runs in direct contradiction to the fact that the road was classified as off-standard.

Table 4.1. Statistical facts of ASD values in highway M-221.

Statistical measure	ASD DTM01	ASD DSM01	ASD DSM MMS
Count	5,118	5,118	5,118
Average	403.3 m	320.5 m	274.4 m
Median	360.0 m	280.0 m	235.0 m
Standard deviation	226.7 m	174.5 m	153.2 m
Coefficient of variation	56.2 %	54.5 %	55.8 %
Minimum	80 m	60 m	55 m
Maximum	1,195 m	965 m	955 m
% length ASD < SSD	3.24 %	5.88 %	9.09 %

Table 4.2. Statistical facts of ASD values in highway M-325.

Statistical measure	ASD DTM01	ASD DSM01	ASD DSM MMS
Count	5,735	5,735	5,735
Average	250.2 m	218.1 m	206.9 m
Median	190.0 m	155.0 m	150.0 m
Standard deviation	189.5 m	180.6 m	162.8 m
Coefficient of variation	75.7 %	82.8 %	78.7 %
Minimum	25.0 m	20.0 m	15.0 m
Maximum	1,095 m	895 m	845 m
% length ASD < SSD	0.65 %	2.06 %	1.90 %

As stations were set 5 m apart, the ASD values will be multiples of 5 m. It is worth recalling that these ArcMap *Feature classes* remained unchanged for each corresponding study. Therefore, in order to determine whether it might be considered that the three series of results statistically come from the same distribution, a non-parametric test for paired samples is needed. Hence a Wilcoxon signed rank test was performed to compare the median values in pairs. Also, the root mean square (RMS) of the resulting ASD sets of values was calculated for each pair according to equation (4.1).

$$RMS = \sqrt{\frac{1}{n} \sum_{i=1}^n (ASD_{q,i} - ASD_{p,i})^2} \quad (4.1)$$

where $ASD_{p,i}$ is the i -th value of the ASD using the elevation model p (the same applies to q with $p \neq q$) and n is the sample size. Finally, the average difference of all pairs of values is calculated.

With regard to the Wilcoxon signed rank test, p-values were much less than 0.05 for each of the three possible elevation model comparison on both highways, so there is a statistically significant difference between all distribution pairs at the 95.0% confidence level (Table 4.3). The average difference provides a quantitative measure of the differences of the paired values,

where it can be observed that the values are large themselves. The RMS removes the effect of the sign in this measure. Although smaller than the ASD standard deviation values, RMS values are large when compared to SSD values and may result significantly different at critical locations.

Table 4.3. Results of the paired samples comparison.

Highway	Parameter	DTM01 vs. DSM01	DTM01 vs. DSM MMS	DSM01 vs. DSM MMS
M-221	W. p-value	<0.001	<0.001	<0.001
	Avg. Diff.	82.9 m	128.9 m	46.1 m
	RMS	173.5 m	221.6 m	113.7 m
M-325	W. p-value	<0.001	<0.001	<0.001
	Avg. Diff.	82.9 m	128.9 m	46.1 m
	RMS	76.7 m	110.4 m	66.1 m

It was observed that the major differences in sight distance between models correspond to areas where elements accumulate by the roadsides, mainly trees. In order to study the influence of such elements in greater detail, three possible scenarios were selected on highway M-325: areas with sparse vegetation, slightly wooded areas and densely wooded areas (with tree crowns hanging partially over the roadway). The results in the three scenarios contemplated are explained in detail in the subsections that follow.

4.3.2.1. Scenario 1: Areas with sparse vegetation

A subsection of highway M-325 with sparse vegetation was selected, from station 3+000 to 3+900 on the outward direction. This subsection is surrounded mainly by low height crops and a few isolated trees, i.e. no relevant sight restrictions besides the bare terrain. On the horizontal projection, the road alignment comprises long tangents and sharp curves on a rolling terrain (Figure 4.5).

Figure 4.5. View of highway M-325 in area with sparse vegetation.

Figure 4.6 shows the results obtained in this subsection. Between stations 3+000 and 3+900, the ASD values obtained for each elevation model match very close, the differences being smaller

than 10 m at any station within this section. This fact supports the hypothesis that, in the absence of dense vegetation or other elements by the roadsides, the ASD outcome for all elevation models is very similar. Minor differences between ASD values using DTM01 and DSM01 models are observed from station 3+730 to 3+780. They may be caused by sparse vegetation and weeds on a cut side slope, detected by the terrestrial LiDAR sensors only.

Figure 4.6. Comparison of ASD in area with sparse vegetation.

4.3.2.2. Scenario 2: Slightly wooded area

In this case, a subsection of road was chosen that had a substantial number of trees on either roadside, but the tree crowns did not overhang the roadway (Figure 4.7). The section runs from station 3+900 to 4+500. The terrain is flatter than in the previous subsection, hence the effect of vegetation is readily apparent.

Figure 4.7. View of highway M-325 in slightly wooded area.

The ASD values obtained using the DTM01 were greater than those of the outcome using either of the DSMs (Figure 4.8). The trees by the inner roadside reduce the ASD in 400 m (approximately the length of the tangent afterwards) over a range of almost 100 m when the DTM01 and the DSMs are compared. However, the ASD values for the two DSMs sets are very similar, differing by 5 m or less. These results were foreseeable and confirm the lowering effect of vegetation on ASD. In sections where elements by the roadsides such as shrubbery or trees, which are not considered by the DTM, the ASD obtained using a DSM will be shorter. Small differences between ASD values on both DSMs (from station 4+300 to 4+500) may be caused by differences in survey sensitiveness of model, its definition and resolution, and their varying adaptation to the environment.

Figure 4.8. Comparison of ASD in slightly wooded area.

4.3.2.3. Scenario 3: densely wooded areas

A subsection of the highway M-325, from station 5+100 to 5+600, was selected to illustrate the ASD differences in a densely wooded area. It has large trees on the roadsides with some of their crowns overhanging the roadway as if they formed a canopy over the pavement surface (Figure 4.9). However, those points were removed as explained in section 3.1.3. In this subsection the alignment is more winding than in the previous ones and there are cut sections by the roadsides.

Figure 4.10 shows the different results obtained using the elevation models proposed. Similar to scenario 2, the resulting ASD using the DTM01 are equal to or greater than those for the DSMs. In contrast to the results for scenario 2, the differences are smaller owing to the additional effect of cut side slopes, which reduce the ASD in all cases. Noteworthy differences in the ASD values between the two DSMs were also found. They may be caused not only by the different resolution or equipment sensitiveness of the models but also due to traffic signs, metallic barriers (for example at station 5+475), etc. The results in this subsection are critical since the ASD values are often near or below the required SSD, hence the need for a reliable sight distance modelling method. In this sense, DSM's provide a more lifelike representation of the highway and its roadsides as it considers the more relevant features.

Figure 4.9. View of highway M-325 in densely wooded area.

Figure 4.10. Comparison of ASD in densely wooded area.

4.4. Concluding remarks

In this chapter, a DTM and two different DSMs were utilized to compute 3D sight distance on the selected highways. The DTM systematically ignores fundamental sight obstructions. Consequently, its use only leads to unbiased results if the lines of sight sway within the roadway and bare terrain areas. The use of a DSM requires that the cloud of points is processed, yet the results are more realistic. Using a DSM instead of a DTM, side obstructions such as vegetation and buildings are taken into account. These obstructions indeed reduce the ASD, produce hidden and reemerged sections or, on the contrary, cover up highway reemerged sections. The high resolution of the MMS DSM facilitates a precise computation of sight distance, yet the reduction of the cloud of points density as the surveyed areas are farther from the sensors is a

disadvantage that must be borne in mind, especially if the lines of sight are expected to traverse areas with low density of points.

The obstruction of lines of sight, due for example to tree trunks or vertical signs, are so short that it may not exert relevant effect on ASD, especially if considering the dynamic nature of visibility while driving. This corresponds to the seen stations shown scattered over the sight distance graphs beyond the ASD. Because the driver as well as the non-seen area move, ASD could be considered longer than that resulting from the models studied. In addition barriers provide orientation of the actual course of the road even though the roadway surface cannot be seen. Therefore, in some cases the real ASD value to consider may be somewhere in between the one provided by the DTM and the DSM (particularly MMS DSM) respective results. An alternative solution to this is the use of gross error removal tool available in *Road Sight Distance*. However, the discretionary use of such a tool is not recommended due to the fact that results would be being altered without regarding the real causes of sight distance interruptions.

The TIN triangulation from a grid produces diagonals in squares randomly since their construction is indifferent according to Delaunay's triangulation criteria. This fact causes that line of sight on the DTM could be evaluated as non-seen whereas the same line is evaluated as seen. It usually occurs in areas where abrupt elevation changes are present, for example cut sides. As a result, the ASD is a few meters greater when evaluated on a DSM than on the corresponding DSM in a particular station even though there is no landscape feature other than the bare ground.

The proportion of way where ASD is shorter than the required SSD by the pertinent standard was used to compare the three sight distance modelling methods. It was, moreover, utilized as an indicator of the alignment design quality in terms of standard sight distance compliance. However, this deterministic criteria has two important drawbacks. First, the design parameters are selected at conservative percentile values, overlooking their stochastic nature. Second, the deterministic criteria does not provide any measure of the impact of deviations from standard. Not only tangential friction, t_p stochastic values are dismissed, but also design speed is considered instead of the actual operating speed. To overcome these disadvantages, a stochastic assessment approach to evaluate the required SSD will be presented in the next chapter.

Chapter 5

RELIABILITY ANALYSIS

5.1. Introduction

In many situations, there are no CMFs available to predict the safety impact of particular road features such as sight distance restrictions. Instead of relying on accident databases, reliability theory has been utilized to provide a surrogate tool in order to evaluate the risk associated with emergency stopping maneuvers. In highway design, the term P_{nc} has been adopted to label the probability of a design not meeting a standard.

The first part of this chapter deals with a pilot study carried out on one of the selected highways to test the feasibility of merging continuous 3D sight distance estimation and reliability analysis broaching almost all variables involved. Also, conditioned probability is contemplated at ease. In the second part, although less variables are contemplated, a greater sample of selected highways is considered, focusing on horizontal curves, which are accident prone locations. By using the FORM, a higher accuracy is achieved while the computational time is reduced. In the third part, the adjustment of SPF's incorporating the reliability measures obtained is presented.

5.2. Pilot study using a sampling method

5.2.1. Methodology

5.2.1.1. Data description

The pilot study aims to analyze the SSD on highway M-607 using a probabilistic approach by means of a Monte Carlo simulation. This highway was selected for the pilot study owing to its generous alignment so that the implementation of the simulation is simpler. As described in section 3.3, this highway link spans 12,284 m, it is located on rolling terrain and its design speed is 100 km/h.

Sight distance may be limited at crest and horizontal curves, locations where pertinent standards set minimum values to ensure long enough ASD. In this selected highway, there are 9 crest curves where the vertical curvature does not comply with the required minimum K_V value of

7,125 m set by the Spanish geometric design standard (Ministerio de Fomento, 2000)³. The AASHTO (2011) criterion is much less restrictive, recommending a minimum of 5,200 m, which is not attained by 2 crests only. The minimum radius in horizontal curves in the selected highway is 450 m. Further details on alignment facts can be found in Appendix I.

In this study, a DSM consisting of a 2.5-m spaced grid (see Table 3.2) was used to shape the road and its roadsides, featuring the terrain as well as vegetation and buildings. As stated in previous chapters of this thesis, DSM's provide a more reliable representation of the road and its environment in terms of sight distance.

As throughout all this thesis, the vehicle paths were traced on each lane as parallel lines, at an offset of 1.5 m from the outer lane edge, in accordance with the former Spanish standard (Ministerio de Fomento, 2000). These paths were discretized into a set of stations spaced 5 m apart. Since the vehicle path is derived from the 3D centerline of the highway, 3D alignments are implicitly regarded, along with the terrain and the landscape features. The ASD was calculated with different observer and target heights. In order to simulate the driver's eye height, the observer heights were fixed at 0.9, 1.1, 1.3, 1.5 m for passenger cars; and 1.8, 2.1 and 2.5 m for heavy vehicles. The target heights were set at 0.2, 0.6 and 1.0 m to simulate different stopping causes. The ASD computations were performed in both driving directions and all possible combinations of both heights were contemplated as inputs.

5.2.1.2. Monte Carlo simulation

A procedure using a Monte Carlo simulation was programmed in MATLAB. The simulation performed a set of 100,000 trials of possible emergency stops along the selected in-service highway. The aim was to check whether simulated ASD were long enough to perform an emergency stop in the event of there being an unexpected object on the pavement surface. Therefore the LSF evaluates the difference between ASD and SSD on each simulated case.

$$LSF = ASD - SSD \quad (5.1)$$

On the one hand, the ASD is computed with the add-in on GIS. The simulation considered that the observer's eye and target height are real random numbers within the intervals described and come from continuous statistical distributions. Thus the simulation script retrieves the ASD values to compute the due value by linear interpolation based on the outcome of the abovementioned combinations, covering all possible values.

On the other hand, the SSD is evaluated according to Equation (2.6) in the German standard (FGSV, 1995b) because this one permits a more comprehensive calculation incorporating the random variables involved in the stopping maneuver.

The workflow of the simulation process is depicted in Figure 5.1. AADT and horizontal alignment data are fixed inputs previously loaded. The driver's eye height (h_1) is the only variable involved both in the ASD and the SSD computation.

³ As explained in a footnote in Chapter 3.3, this highway is unlikely to have been designed in accordance with the current standard (Ministerio de Fomento, 2016), which provides less restrictive requirements for crest vertical curves owing to the target height raise (see Table 2.1).

Figure 5.1. Flowchart of the simulation process.

Each of the components of Figure 5.1 is associated with one or more variables. Table 5.1 summarizes the components and variables considered in the simulation in the order they were generated or computed. It also specifies whether they are independent random, conditioned random or determined by other variables.

Table 5.1. Simulation variables.

ID	Component	Variable	Type	Input variables
1	Kilometer	Kilometer	Random	
2	Direction	Direction	Random	
3	Target height	h_2	Random	
4	AADT data	Vehicle type	Random	
5	Driver's eye height	h_1	Conditioned	Vehicle type
6	ASD	ASD	Determined	Kilometer, h_2 , h_1
7	Operating speed	v_{50}	Determined	Kilometer
8	Operating speed	σ_v	Determined	v_{50}
9	Initial speed	v_0	Determined	Kilometer, v_{50}
10	t_p	t_p	Random	
11	Air resistance	ρ	Random	
12	Vehicle features	C_x	Conditioned	Vehicle type
13	Vehicle features	S	Conditioned	Vehicle type
14	Vehicle features	W	Conditioned	Vehicle type
15	Air resistance	F_a	Determined	ρ , C_x , S , W
16	Vertical alignment	G	Determined	Kilometer, direction
17	Tangential friction	F_t	Determined	v along stopping
18	SSD	SSD	Determined	v_0 , t_p , F_t , G , F_a , W

The first two values to be generated were kilometer and direction. They were generated according to a uniform distribution in the range of the sum of both distances. It is worth noting that both ends of the highway link were dismissed from a distance that ranges the stations from where the final station could be seen with maximum values of h_1 and h_2 because, at the highway link ends, all the ASD equal the distance to the final station.

At each of these locations, random target heights ranging from 0.2 to 1.0 m were then generated according to a uniform distribution. The vehicle type is assigned randomly using a binomial distribution based on AADT composition, distinguishing between passenger cars and heavy vehicles. In the section studied, the AADT was 14,014 vehicles per day with 12.89 % of heavy vehicles (Comunidad de Madrid, 2014). Next, the corresponding value of h_1 is generated from different distributions depending on the vehicle type. Two different distributions are used for passenger cars and heavy vehicles. This distinction also serves to generate values for the parameters involved in the estimation of air resistance: drag coefficient C_x and the reference area of the vehicle master section S .

The ASD value corresponding to each simulated case is calculated by linear interpolation of the tabulated values that resulted from the 3D ASD computation on GIS. The base ASD interpolation values were those at the immediately prior and following stations of both observer and target positions, and upper and lower heights of both h_1 and h_2 , these inputs leading to a single ASD value.

Once the ASD of each particular case is determined, the second part of the process is launched to compute the simulated SSD. In this part, alignment data are needed. On the one hand, horizontal alignment is necessary for initial vehicle speed estimation. It was also used, as previously mentioned, to determine the vehicle path (as per subsection 3.1.3.2). On the other hand, the integration of the SSD equation requires vertical alignment data, in order to determine the grade values along the stopping trajectory. Moreover, the speed profile of heavy vehicles is deduced from the vertical alignment.

Next, the process includes the simulation of the driver braking performance. The t_p and the initial speed v_0 are generated. As stated in the literature review, the distribution of t_p depends on the driver's abilities. The results of Lerner (1995), which stated that this variable follows a lognormal distribution where the mean is 1.5 s and the standard deviation is 0.4 s, lead to a long tail on the right side, producing many possible outliers. On this basis, the simulated t_p value sample is assumed to follow, in a first approximation, a normal distribution, whose mean was 1.5 s and standard deviation was 0.3 s, such that approximately 95% of t_p values are below 2 s.

Initial vehicle speed is assigned to passenger cars according to the speed prediction model described by Castro et al. (2006a). The speeds of this model are assumed to depend on the horizontal curve radius. It is worthy of note that it was developed by studying operating speeds on this same section of highway, amongst others. Unlike other geometric design research areas, where the parameter most frequently used is 85-th percentile speed, here it is necessary to use 50-th percentile speed to characterize the statistical distribution of values:

$$v_{50} = 105.35 - \frac{4541.98}{|R|} \quad (5.2)$$

where R is the radius of the horizontal curve. The model assumes that drivers, on average, achieve the desired speed in long enough tangents with a constant acceleration and deceleration rate of 0.85 m/s^2 as the range of radii yields speed values on curves close to the desired speed. According to the model, the standard deviation of speed also depends on the radius and can be formulated as follows:

$$\sigma_v = 17.03 - \frac{111.25}{\sqrt{|R|}} \quad (5.3)$$

Hence after the exact v_{50} value is calculated, a random variation is added. A normal distribution is assumed for the estimation of speed randomness in the simulation. This randomness has been extrapolated all along the section by combining Equations (5.2) and (5.3):

$$\sigma_v = 17.03 - 1.65074 \sqrt{105.35 - v_{50}} \quad (5.4)$$

The assumed desired speed on tangents was 105.35 km/h according to the limiting value in Equation (5.2). A minimum threshold value of 40 km/h is assumed for the speed model. However, since such cases are not likely to yield non-compliant stopping maneuvers, tails of low speeds are unlikely to exert any influence on the experiment. For this reason, the lower tail of speed samples have not been truncated.

Considering heavy vehicles has several implications for the model. The simulation assumed that heavy vehicle initial speeds come from different distributions, which comprises both trucks and buses. Dey et al. (2006) studied bimodal speed distribution curves. Donell et al. (2001) developed a speed prediction model for heavy vehicles, finding that vertical alignment has a greater influence on heavy vehicle speed than horizontal alignment. Based on this, a speed prediction model has been built according to acceleration and deceleration curves for heavy vehicles which was deduced from the acceleration and deceleration curves in the Spanish standards (Ministerio de Fomento, 2016) (see Figure 2.25). A study also showed that the average speed was 83 km/h on Spanish two-lane rural highways (Castillo, 2008). Thus in this research, 83 km/h was considered the desired speed for heavy vehicles. The grades in this section are short enough to allow the consideration of linear speed variation per distance travelled on grades. A critical grade value is associated with each speed. When the critical grade is exceeded, the heavy vehicle is assumed to start a decelerating segment where the deceleration rate depends on the grade. The deceleration segment finishes where the grade falls below the critical grade again. Then the heavy vehicle is assumed to accelerate until the desired speed of 83 km/h is reached or until the critical grade is again exceeded. The critical grade depends on the speed and can be defined as the asymptotic speed that can be achieved by a heavy vehicle after a long enough tangent on such a grade. Random variation in the speed sample generated has been added also according to standard deviation in Equation (5.4). Speed values above 102 km/h for heavy vehicles were truncated. Figure 5.2 shows an example of the v_{50} speed profile for passenger cars and heavy vehicles on a subsection of the selected highway. Whereas the speed for passenger cars is affected by horizontal bends, the heavy vehicle speed changes according to the existing grades.

Figure 5.2. 50-th percentile speed profiles in a subsection of highway M-607.

The next step is braking simulation. To estimate braking distances, the model must consider initial vehicle speed, air resistance, vehicle inertia and tangential friction. It is recalled that, the tangential friction is modelled by German RAS-L (FGSV, 1995b) as a polynomic function depending on the speed (see Equation (2.6)). Due to the lack of references in literature on its variance, no uncertainty has been associated with this. Also, air resistance r_a is estimated by dividing aerodynamic friction f_a by the mass of the vehicle m :

$$r_a = \frac{f_a}{m} \quad (5.5)$$

where aerodynamic friction can be calculated according to Equation (2.5). The air density (ρ) was assumed variable to simulate different meteorological conditions such as wind and atmospheric air pressure. C_x is the drag coefficient of the vehicle. Important differences arise in this respect in the case of heavy vehicles. Whilst it ranges from 0.27 to 0.40 approximately in passenger cars, in heavy vehicles this coefficient fluctuates between 0.6 and 0.7 (Hucho, 1987). The simulation has adopted these ranges using normal distributions. S is the reference area of the vehicle master section, which depends on the type of vehicle: in passenger cars, the reference area is approximately 2 m² while in heavy vehicles this value is ca. 6.5 m².

Next, a numerical integration procedure based in a finite element method has been included in the simulation to calculate the required SSD (Equation (2.6)). Through the rectangle method, the function is evaluated at intervals of 1 km/h to finally retrieve the total SSD length. The integration procedure starts at the moment the driver is supposed to apply the brakes, i.e. after having travelled at constant speed v_0 the distance which corresponds to t_p . Figure 5.3 sketches the iterative process, in which the result of the integral of Equation (2.6) in step i is used in step $i+1$. The triangles on the longitudinal profile mark the positions where the vehicle is placed during iterations. Since the algorithm first checks where the vehicle is placed to assign the grade value at that position according to vertical alignment data, it is necessary to retrieve the distance travelled in step i . Once the loop reaches the final speed $v = 0$, the overall distance travelled in

braking is stored. The sum of the distance travelled during t_p and the total braking distance is the required SSD.

Figure 5.3. Numerical integration of SSD equation.

Finally, the program retrieves each value of braking distance to check whether ASD is exceeded by SSD. The margin of compliance can be defined as the result of the LSF, i.e. the difference between ASD and SSD. An unfavorable situation occurs whenever required SSD is greater than ASD, i.e. when the margin of compliance is below 0. The overall number of such unfavorable cases represents the probability of non-compliance (P_{nc}). However, it must be emphasized that an unfavorable case does not necessarily lead to a collision since the driver could make evasive manoeuvres (Zheng, 1997) rather than crashing, if the circumstances so permit.

5.2.2. Results and discussion

In the sight distance computation, which is prior to the Monte Carlo simulation, it is possible to analyze the average effect of height inputs in the highway studied. This is interesting, for example, to measure the predicted impact of the new geometric design standard value for the target height in the Spanish instruction ($h_2 = 0.5$ m) in comparison with the previous one ($h_2 = 0.2$ m). Figure 5.4 exhibits the average ASD values that resulted from all the combinations of driver's eye height and target height in the outcome sets. Provided that the driver's eye height remains constant at 1.1 m, the average increase of ASD is 16.1 m.

The evaluation of the difference between ASD and SSD was performed in 100,000 trials by means of a Monte Carlo simulation. The results indicated that the overall P_{nc} on the highway was 12%. The histogram of the margin of compliance is displayed in Figure 5.5, where non-compliance cases are highlighted in red.

Figure 5.6 shows the density distribution resulting from the simulation depending on the initial speed and the margin of compliance. The scale bar indicates the number of simulated cases in the area. It can be noticed that the higher the initial speed is, the higher the rate of non-compliance cases is and the shorter the margin of compliance is.

Figure 5.4. Average ASD depending on the input heights.

Figure 5.5. Margin of compliance histogram.

Next, reliability measures are presented as per equations (2.44), (2.45) and (2.46). According to the simulation outcome, the P_{nc} along the highway link for heavy vehicles was 2.6 %. It can, therefore, be affirmed that the P_{nc} is significantly lower for heavy vehicles than for passenger cars (13.4%). Consequently, the reliability index β was higher for heavy vehicles than for passenger cars. This fact supports the usual assumptions in geometric design which only

considers passenger cars when studying ASD on highways. The results of reliability analysis as per equations (2.43) to (2.46) can be found in Table 5.2. The parameter which makes the P_{nc} lower for heavy vehicles is the significantly lower mean of the required SSD.

Figure 5.6. Margin of compliance density map.

Table 5.2. Results of ASD and reliability analysis.

Dataset	ASD [m]		SSD [m]		Reliability measures	
	Mean	St.Dev.	Mean	St.Dev.	P_{nc}	β
Passenger cars	309.69	151.75	172.21	41.58	0.134	0.887
Heavy vehicles	327.87	154.11	132.54	30.33	0.026	1.264
All vehicles	312.06	152.18	167.04	42.44	0.120	0.929

Afterwards, the highway link was divided into 50-meter segments in which the P_{nc} was in detail for results in both directions. As a result, areas of higher hazard were identified along the section. The results showed that there are five clusters of segments where the P_{nc} exceeded 38%. Three of these clusters are located within the horizontal curves of minimum radius (450 m), two of them coinciding partially with crest curves ($K_V = -6,000$ m and $K_V = -5,700$ m). Figure 5.7 shows the probability of hazard in segments along a specific zone of the section studied. The first of the aforementioned clusters can be seen at station 0+500 m from the starting point, at the beginning of the crest curve. The high P_{nc} values produced at station 2+050 m can be attributed to the existing horizontal curve of minimum radius, since there is a constant grade in vertical alignment. It can also be seen in Figure 5.7 that patterns are somewhat similar for passenger cars and for heavy vehicles. Another cluster of high P_{nc} is located on a tangent where the grade reaches 6%. These results clearly show the relation between the tightest alignments and the potential accident prone locations.

Figure 5.7. P_{nc} after segmentation and vertical alignment.

5.3. Study on curves using RT software

The goal pursued in this section is to understand the safety implications of modelling ASD by means of reliability analysis, particularly, to predict the safety impact of sight distance restrictions in curves. The ASD and the required SSD of 402 horizontal curves, located in the selected highways, is compared through a probabilistic approach. The selected curves were classified into different groups based on the existing roadside features or geometric elements that hinder driver's visibility. Three ASD estimation methods were used which include a 2D method and a 3D method using two different elevation models. The ASD results obtained through 2D and 3D methodologies are compared. The different conditions under which 3D ASD estimation is important, were identified. Next, reliability theory is utilized to evaluate the risk level (P_{nc}) associated with limited sight distance for each ASD modelling method. Finally, the P_{nc} results were compared in order to investigate the significance of the safety implications of the ASD modelling approach.

In the next subsections, the inputs and procedures that apply only to this part are described, namely the determination of the minimum ASD at each curve over the three different sight distance modelling methods, the operating speed profile construction, and the characterization of the random variables considered.

5.3.1. Determination of available sight distance input values

Three ASD modelling methods were used in this study, a 2D method and two 3D methods. On each of the three methods, the geometric parameters set by the former Spanish standard (Ministerio de Fomento, 2000) with regard to driver's eye height, target height and path location on the cross section were used.

Next, this reliability analysis compares the P_{nc} values from the three different ASD modelling methods. It is necessary to determine the input values of each model to yield the corresponding P_{nc} and the worst case is produced by the minimum ASD. The next sections describe the process of obtaining such ASD and P_{nc} values.

5.3.1.1. 2D horizontal available sight distance

The minimum 2D ASD was obtained in CAD software on the horizontal projection for all the curves in both directions. The circular arc of some curves is often shorter than the ASD, therefore the ASD may be longer than the value resulting from Equation (5.6) using the respective cross sectional values (derived from equation (2.9), see Figure 2.2):

$$ASD = 2 \cdot (R - l + b) \cdot \cos^{-1} \left(\frac{R - l_w - C}{R - l_w + b} \right) \quad (5.6)$$

where:

- C = minimum horizontal offset from the sight obstruction to the inner roadway edge, m.
- R = curve radius, m.
- l_w = lane width, m.
- b = offset between the driver and the inner roadway edge, m.

Some of the values obtained are shown in Figure 5.8 to illustrate the statement above. The boundary represented by Equation (5.6) differs slightly on each highway link as their corresponding cross section dimensions differ.

5.3.1.2. 2D vertical available sight distance

In this part, the ASD on crests has been estimated analytically. There are two possible cases. On the one hand, if the ASD is longer than the crest length, the minimum ASD is constant while both the observer and the target are located within the crest segment (Figure 5.9). Station s_1 is the inbound end of the crest bend and station s_2 is the outbound end of the crest after subtracting the ASD value.

The ASD is given by the following equation on long crests:

$$ASD = \sqrt{2 \cdot K_V} \cdot (\sqrt{h_1} + \sqrt{h_2}) \quad (5.7)$$

Figure 5.8. Comparison of 2D horizontal ASD and curve radius in sample.

Figure 5.9. ASD graph on a crest longer than ASD.

The most unfavorable case for the failure mode considered in reliability analysis occurs where the target is at the stitching point between the crest and the outbound grade (red sketch in Figure 5.10) because the grade resulting between observer and target positions is minimum, which yields a longer SSD.

Figure 5.10. Lines of sight on a crest longer than ASD.

On the other hand, the ASD may result shorter than the crest length. In such a case, at least either the observer or the target are outside the crest where the line of sight yields minimum ASD.

Figure 5.11. Parameters defining the line of sight on a crest shorter than ASD.

According to Figure 5.11, the ASD on the vertical curve is given by:

$$ASD = x + L + u \quad (5.8)$$

where:

$$h_1 = \frac{t^2}{2 \cdot K_v} + \frac{t}{K} \cdot x \Rightarrow x = \frac{2 \cdot K_v \cdot h_1 - t^2}{2 \cdot t} \quad (5.9)$$

$$h_2 = \frac{(L-t)^2}{2 \cdot K_v} + \frac{(L-t)}{K} \cdot u \Rightarrow u = \frac{2 \cdot K_v \cdot h_2 - (L-t)^2}{2(L-t)} \quad (5.10)$$

The variable t defines unequivocally the position of both at the point of minimum ASD. To find local minima, the derivative of ASD with respect to t is computed and made equal to 0.

$$\frac{\partial ASD}{\partial t} = \frac{K_v \cdot [h_2 \cdot t^2 - h_1 \cdot (L-t)^2]}{t^2 \cdot (L-t)^2} = 0 \quad (5.11)$$

If the equation is resolved assuming that $K_v \neq 0$ and $h_1 \neq h_2$, two solutions arise. However, the value that yields the minimum ASD is the one with the negative sign:

$$t = \frac{\sqrt{h_1} \cdot L}{\sqrt{h_1} \pm \sqrt{h_2}} \quad (5.12)$$

Particularizing for $h_1 = 1.1$ m and $h_2 = 0.2$ m:

$$t = 0.74338 \cdot L \Rightarrow \begin{cases} x = \frac{1.47973 \cdot K_v}{L} - 0.37169 \cdot L \\ u = \frac{0.77936 \cdot K_v}{L} - 0.12831 \cdot L \end{cases} \quad (5.13)$$

The minimum ASD on short crests is therefore:

$$ASD = -\frac{K_v}{L} \left(\sqrt{h_1} + \sqrt{h_2} \right)^2 + \frac{L}{2} \quad (5.14)$$

5.3.1.3. 3D available sight distance

The minimum ASD for horizontal curve is a local minimum located around the curve (Figure 5.12). Such a minimum does not necessarily lay within the bounds of the curve. The inverse ASD provides a good reference to determine whether the lines of sight associated to the minima indeed orbit around the curve, i.e. the minima are caused by either the alignment or the features by the roadside at the curve location. In the case of the curve in Figure 5.12, the minimum ASD is produced at different stations when computing sight distance using a DTM or a DSM. Whilst the minimum ASD with the DTM is located at station 942 (ASD equals 74 m), the corresponding station for the DSM is 0+949 (ASD equals 72 m). Besides different ASD values, it must be also highlighted that these two stations may correspond to different operating speeds on the speed

profile. As it will be discussed later in this chapter, here lies part of the effect produced by the sight distance modelling method on safety analysis (P_{nc}).

Figure 5.12. ASD and inverse ASD around a horizontal curve on highway M-325.

The complete results of the ASD calculation are available in Appendix II (digital version only).

5.3.2. Operating speed model

In this subsection, the procedure followed to build up the operating speed profile of the selected highway links featured in section 3.3 is described. This speed profile ought to deliver a realistic prediction of the range of speeds selected by drivers at every position along the highway links. The model assumptions contemplate curves, tangents, intersections such as roundabouts that must be driven by the vehicles travelling along the highway, and eventually transition to urban sections. In what follows, the assumptions made to assign operating speeds over the selected highways are outlined.

Instead of using the 85-th percentile speed, the mean speed at every point is required in this study since speed will be dealt with as a random variable. The mean speed is, as a matter of fact, the easiest statistical parameter to handle, yet other parameters such as statistical measures of dispersion (i.e. the standard deviation) will also be required. In addition, the assumptions to be made must yield an unbiased speed profile for the range of parameters and features existing in the selected highway links.

5.3.2.1. Acceleration and deceleration rates

As seen in the review of operating speed models review (Section 2.5), many theoretical models assume constant acceleration and deceleration rates. Unlike it is shown in most literature

exhibits, the resulting acceleration curves when plotting speed - distance graphs are quadratic parabolas. Moreover, this assumption runs against the principles of dynamics because the higher the speed is, the higher the aerodynamic friction will be. For example, the former German standard (FGSV, 1995b) modelled the ratio of air resistance to weight in SSD, i.e. deceleration of a passenger car, as a function of the square of speed (Equation (2.8)). The literature review (Section 2.5.3) showed that variable acceleration and deceleration rates are assumed in different models, more according to the laws of dynamics. Unlike in section 5.2, where the speed transitions are short, acceleration and deceleration rates are here assumed to be variable. Like in the TWOPAS model, both rates will be assumed to respectively vary linearly over distance as a function of the speed:

$$a(v) = A + Bv \quad (5.15)$$

This assumption simplifies the considerations to sketch the speed profile as they define an unambiguous curve because speed variations not only occur on tangent-to-curve transitions but also take place between two circular arcs. The equations presented henceforth aim to provide a practical tool for the sketch of speed transitions over the highway stationing. Based on the literature review (Section 2.5), the boundary conditions are assumed to determine the acceleration expression in accordance with the following linear equation system:

$$\left. \begin{aligned} A_a + \frac{105}{3.6} \cdot B_a &= 0.5 \text{ m/s}^2 \\ A_a + \frac{105}{3.6} \cdot B_a &= 1.1 \text{ m/s}^2 \end{aligned} \right\} \begin{aligned} A_a &= 1.24118 \text{ m/s}^2 \\ B_a &= -0.0254118 \text{ s}^{-2} \end{aligned} \quad (5.16)$$

Similarly, the boundary conditions to determine the deceleration rate expression are according to the Equation system (5.17). Also supported by literature (Pérez Zuriaga, 2012), it assumes that deceleration rates are, in absolute value, significantly higher than the acceleration rates.

$$\left. \begin{aligned} A_a + \frac{105}{3.6} \cdot B_a &= -0.6 \text{ m/s}^2 \\ A_a + \frac{105}{3.6} \cdot B_a &= -1.3 \text{ m/s}^2 \end{aligned} \right\} \begin{aligned} A_a &= -1.46471 \text{ m/s}^2 \\ B_a &= 0.0296471 \text{ s}^{-2} \end{aligned} \quad (5.17)$$

Either for acceleration and deceleration, the equation of the speed can be deduced as follows:

$$a(v) = \frac{dv}{dt} \Rightarrow dt = \frac{dv}{a(v)} \Rightarrow \int_0^t dt = \int_{v_0}^v \frac{dv}{A + Bv} = \left[\frac{\log(A + Bv)}{B} \right]_{v_0}^v + C \quad (5.18)$$

The constant can be set equal to zero without loss of generality, resulting:

$$t = \frac{1}{B} \log \frac{A + Bv}{A + Bv_0} \Rightarrow e^{Bt} = \frac{A + Bv}{A + Bv_0} \Rightarrow v(t) = \left(\frac{A}{B} + v_0 \right) \cdot e^{Bt} - \frac{A}{B} \quad (5.19)$$

Similarly, the distance traveled can be deduced as follows:

$$v(t) = \frac{dx}{dt} \Rightarrow dx = v(t) \cdot dt \Rightarrow \int_{x_0}^x dx = \int_0^t \left[\left(\frac{A}{B} + v_0 \right) \cdot e^{B \cdot t} - \frac{A}{B} \right] \cdot dt \quad (5.20)$$

$$x - x_0 = \left[\frac{A + B \cdot v_0}{B^2} \cdot e^{B \cdot t} - \frac{A}{B} \cdot t \right]_0^t = \frac{A + B \cdot v_0}{B^2} (e^{B \cdot t} - 1) - \frac{A}{B} \cdot t \quad (5.21)$$

Substituting the expression of t in Equation (5.19):

$$x - x_0 = \frac{v - v_0}{B} - \frac{A}{B^2} \cdot \log \frac{A + B \cdot v}{A + B \cdot v_0} \quad (5.22)$$

Figure 5.13 compares the speed profile produced by different acceleration models from initial speed 20 km/h. The slope of the black solid line, given by Equation (5.22), decreases over distance more quickly than those of the other curves with constant acceleration rate. Likewise, Figure 5.14 plots the theoretical deceleration curve developed here against constant-deceleration curves, all starting at 105 km/h.

Figure 5.13. Speed profile of different acceleration models over distance.

5.3.2.2. Speed on curves

As previously mentioned, the speed is treated as a random variable. To define the speed distribution that better predicts the values chosen by drivers, at least a measure of central tendency and a measure of deviation are needed. According to the review in section 2.5, models that contemplate the curve radius providing mean values and the standard deviation are available. In addition, the speed prediction through the radius results more robust than the CCR

against error in alignment recreation since the CCR value depends upon the length of the curve. Particularly, the model of Castro et al. (2006a) was developed on some of the selected highway links. It was however devised through speed measurements at a reduced number of curves, within a defined range of radii. Its mathematical expression was given in Equation (5.2).

Figure 5.14. Speed profile of different deceleration models over distance.

Indeed, the literature review showed that it is at least doubtful that such an expression could predict accurately the operating speed on short-radius curves (see Figure 2.26 in section 2.5). For that reason, the mathematical expression adjusted by Pérez-Zuriaga (2012) was utilized to predict more accurately the operating speed on sharp curves (Equation (5.23)). It also uses the radius as the explanatory variable to predict the 50-th percentile speed. Other piecewise-defined speed prediction models are found in literature (Camacho-Torregrosa et al., 2013).

$$v_{50} = \sqrt{2420.63 \cdot \log |R| - 7619.89} \quad (5.23)$$

The intersection of both curves (Equations (5.2) and (5.23)) was the boundary considered to apply either, which is found at the value $R = 97.91$ m. This way, the speed prediction curve is mathematically continuous. To characterize the speed distribution, the standard deviation is needed. Each of the above described expressions is coupled with a standard deviation expression, which is also a function of the radius. On each subdomain, the corresponding standard deviation will be taken, and are respectively given by the equations that follow.

$$\sigma_{v,50} = 17.03 - \frac{111.25}{\sqrt{|R|}} \quad \text{if } |R| \geq 97.91 \text{ m} \quad (5.24)$$

$$\sigma_{v,50} = \sqrt{14.8194 + 5.38444 \cdot \sqrt{|R|}} \quad \text{if } |R| < 97.91 \text{ m} \quad (5.25)$$

5.3.2.3. Desired speed on tangents

Desired speed is the cruise speed adopted by drivers on long tangents. Whether the tangent length is long enough or not to reach such a speed is determined by the speed profile sketch itself.

Based on previous research (Cardoso, 1995; McLean, 1981) the highway features exhibited in Table 5.3 were assumed to obtain the corresponding 50-th percentile desired speeds.

Table 5.3. Desired speed on the selected highway links.

Highway	Terrain type	Design speed [km/h]	Traveled-way width [m]	Desired speed v_{50} [km/h]
M-104	Rolling	60	6.5/7.5	92.38
M-221 / M-222	Rolling	80	6/7	94.91
M-325	Rolling	< 40	5.5/5.5	82.88
M-513	Rolling	40	7/9	94.91
M-600	Hilly	60	7/9	99.41
M-601	Rolling	60	7/9	99.41
	Hilly	40	6.5/7.5	85.35
M-607	Rolling	100	7/12	105.35
M-610	Rolling	40	6.5/7.5	88.87
M-611	Rolling	40	6.5/7.5	82.88
M-629	Mountainous	< 40	5/5	70.35
M-633	Rolling	40	5.5/6	78.87

5.3.3. Limit-state function

In the context of this study, the LSF is the difference between ASD (supply) and SSD (demand), the first one being calculated in three different ways as described in section 5.3.1. It is recalled that, according to the Spanish standard (Ministerio de Fomento, 2016), the SSD is evaluated using the expression in equation (2.2).

Unlike preceding studies (Essa et al., 2016; Hussein et al., 2014), the effect of grade is regarded here. The elevation difference between the station where the ASD takes the minimum value and that of the farthest point viewed from that position, divided by that distance, is taken as the grade value, regardless of the ASD estimation method. In this way, conditions where 2D ASD value is similar to 3D ASD and where they are different could be identified. In the cases where the ASD remains constant along a certain stretch, the worst case, namely the minimum grade, is considered.

5.3.4. Random variables

Based on equations (5.1) and (2.2), the reliability analysis considers three random variables; the initial speed (v), the perception and reaction time (t_p); and the tangential friction factor (f_t). The other two variables, the ASD and the grade (G), are assumed to be deterministic. As mentioned earlier, the speed considered in the analysis of this paper is the operating speed which is

assumed to follow a normal distribution with mean as per equations (5.2) and (5.23), the standard deviation being estimated by equations (5.24) and (5.25). Along each highway segment, the operating speed is estimated at the minimum ASD station. The assumptions concerning t_p used in this study come from the results of an experiment accomplished by Lerner (1995). These assumptions are also taken as reference for perception and reaction time distribution for the National Cooperative Highway Research Program (NCHRP) (Fitzpatrick and Wooldridge, 2001). Therefore, the t_p is assumed to follow a lognormal distribution with a mean of 1.5 seconds and a standard deviation of 0.4 second.

The longitudinal friction factor was assumed to follow a normal distribution with a mean and standard deviation that are dependent on the operating speed as shown in Table 5.4 (Richl and Sayed, 2006). For any speed value that is out of Table 5.4, linear interpolation is performed to estimate the mean and the standard deviation of its corresponding friction factor. The wet surface condition was contemplated as it represents the worst-case scenario for the f_l .

Table 5.4. Coefficients of friction (Richl and Sayed, 2006).

Pavement condition	Mean speed (km/h)	Mean	Standard deviation
Wet	80.4	0.4192	0.0913
Wet	85.0	0.4013	0.0913
Wet	90.0	0.3826	0.0913
Wet	95.0	0.3571	0.0913
Wet	99.8	0.3498	0.0913
Dry	All speeds	0.8852	0.0949

5.3.5. Probability of non-compliance computation

In this part, the FORM reliability method is selected for the analysis. This method was described in detail in section 2.7.1.1. The main concept of FORM is a transformation from the original space of random variables x to the standard normal space of random variables y . In the standard normal space y , the LSF is linearized at the point closest to the origin. This point is called the design point, or the most probable failure point. Subsequently, the design point is the solution to the optimization problem in (0) (Haukaas, 2012a):

$$y^* = \arg \min \{ \|y\| \mid G(y) = 0 \} \quad (5.26)$$

where y^* is the design point coordinates and $G(y)$ is the LSF in the standard normal space. By solving this optimization problem, the distance from the origin in the standard normal space to the design point (y^*) is the reliability index β . Finally, the probability of failure (noncompliance) can be determined by using Equation (5.26), where Φ is the standard normal cumulative distribution function (Haukaas, 2012c).

$$P_{nc} = \Phi(-\beta) \quad (5.27)$$

Rt software was used to compute the P_{nc} value of the LSF shown in Equation (5.1). Rt is a computer program for system reliability and optimization analysis developed by Mahsuli and

Haukaas (2012) at the University of British Columbia, Vancouver, Canada. Also, a MATLAB script was developed to automate the launching process of Rt to compute the P_{nc} several times based on the different inputs described and to retrieve the resulting outputs in a spreadsheet for analysis.

5.3.6. Results and discussion

5.3.6.1. Available sight distance modelling

The ASD was calculated along the 12 selected highway links in both directions. 402 curves were analyzed in both directions (804 cases) and using the three different ASD modelling methods described in thesis (section 5.3.1): detached 2D (2D ASD), 3D on a DTM (3D DTM), and 3D on a DSM (3D DSM). Furthermore, the 804 selected cases were classified into 4 groups based on the existing roadside features or geometric elements that hinder driver's visibility. The four groups are as follows: 1) curves with vegetation by the roadside; 2) horizontal curves with cut side; 3) horizontal curves on a crest vertical curve; and 4) horizontal curves with none of the aforementioned features (i.e. clear highway segments). Out of the overall sample size (804 cases), the first group (vegetation) represents 463 cases, the second group (cut side) represents 147 cases, the third group (crest vertical curve) represents 129 cases, and the remaining 65 cases belong to the fourth group (clear horizontal curves). Some curves may have different obstructions depending on the direction considered. For these curves, cut side or vegetation are typically the limiting elements by the inner roadside on the direction in which the curve bends to the right, whereas a crest vertical curve is the sight restriction on the opposite direction.

A first comparison of the ASD results showed that the 3D DTM ASD is equal or greater than 3D DSM ASD in all cases. This is explained by the effect of the extra elements considered only in DSM method that reduce the ASD, as was expected. A *t*-test for paired samples was carried out to investigate the significance of differences between the three ASD modelling methods. Three pairs were considered in *t*-test: (2D - 3D DTM), (2D - 3D DSM), and (3D DTM - 3D DSM). Table 5.5 outlines the results of this comparison. The *p*-values that correspond to pairs in which the difference was found to be significant at 95 % confidence level are shown in bold.

For the whole sample, the results show that the difference is not significant only for the pair (2D – 3D DSM). The trend of the estimated ASD shows that 3D DTM tends to overestimate the ASD comparing to the other two methods. The mean difference in the pair (2D – 3D DSM) is small; however, the results are still affected by high variances.

With regard to the first group (i.e. curves with vegetation by the roadside), a significant difference is identified for all pairs. Again, the 3D DTM overestimates the ASD comparing to the other two methods. The significant difference for the pair (2D – 3D DSM) can be explained by the 3D approach. That is, the concurrence of both alignment projections (i.e. horizontal and vertical projections) and the vegetation make the 2D approach overestimates the ASD when compared with the 3D DSM.

In the second group (i.e. horizontal curves with cut side), only the pair that compares the two 3D methods has a significant difference. The difference is not significant for the other two pairs

owing to the greater variance. Also, the mean difference in the pair (2D – 3D DTM) is almost negligible, although the variance reaches 24.5 m.

Table 5.5. Comparison of ASD values among the three models.

Group	ASD pair	Mean (m)	St. Dev. (m)	p-value
All cases	2D – 3D DTM	-47.4	100.3	<0.001
	2D – 3D DSM	1.5	48.6	0.372
	3D DTM – 3D DSM	48.9	87.0	<0.001
1) Vegetation	2D – 3D DTM	-69.1	111.6	<0.001
	2D – 3D DSM	10.2	34.5	<0.001
	3D DTM – 3D DSM	79.2	102.6	<0.001
2) Cut side	2D – 3D DTM	-0.1	24.6	0.949
	2D – 3D DSM	3.3	23.9	0.091
	3D DTM – 3D DSM	3.5	3.9	<0.001
3) Crest	2D – 3D DTM	7.0	29.9	0.008
	2D – 3D DSM	7.0	29.9	0.008
	3D DTM – 3D DSM	0.0	0.0	1*
4) Clear	2D – 3D DTM	-108.0	122.9	<0.001
	2D – 3D DSM	-75.0	106.7	<0.001
	3D DTM – 3D DSM	33.0	51.6	<0.001

* The correlation coefficient since the identical samples cannot be compared by the t-test.

For the third group (i.e. horizontal curves on a crest vertical curve), the difference is significant for both (2D – 3D) pairs. The case of (3D DTM – 3D DSM) cannot be compared by the t-test since the ASD values are the same. This occurs because the line of sight that determines the minimum ASD lies within the roadway boundaries, and both 3D models are identical within such area. The resulting Pearson correlation coefficient equals 1, and the mean difference equals 0. The 2D method overestimates the ASD compared to the 3D methods. Presumably, this is due to the 3D features that are ignored by the 2D approach such as the effect of superelevation on the geometry, the existence of other near vertical curves, or the overlapping with a crest vertical curve. This emphasizes the importance of the 3D ASD modelling.

For the last group (i.e. clear highway segments), the results do not follow such a definite pattern as their counterparts do in the other groups. This group is the most heterogeneous as it presents the greatest differences. The clear inner-roadside allows a longer ASD in both 3D models. Moreover, the ASD values depended upon features located before or beyond the curve studied. In all pairs, a significant difference is identified. The 2D model greatly underestimates the ASD when compared to the other two 3D models. The 3D DTM overestimates the ASD in comparison with the 3D DSM model, as can be expected.

Overall, the ASD results show that there is a significant difference among the three ASD modelling methods in most cases. This emphasizes the importance of using the 3D modelled sight distance, particularly the 3D DSM ASD, either in highway design or during the service life as it is considered more realistic than the 2D one.

5.3.6.2. Effect of the sight distance modelling approach on safety

Considering the three ASD modelling methods and the four-groups classification described in the previous subsection, reliability analysis was carried out to evaluate the P_{nc} associated with each case of the selected 804 highway segments. Furthermore, another classification was considered by assuming two variations: flat profile ($G = 0$) and the real grade on the way travelled by the vehicle along the hypothetical emergency stop. T -test for paired samples was conducted to evaluate the differences between the three ASD modelling methods in terms of their corresponding P_{nc} using the same three pairs: (2D - 3D DTM), (2D - 3D DSM), and (3D DTM - 3D DSM). Table 5.6 summarizes the results of this comparison. The p-values that correspond to pairs in which the difference was found to be significant at 95 % confidence level are shown in bold.

Table 5.6. Comparison of P_{nc} values among the three models under both profile assumptions.

Group	P_{nc} pair	Flat profile			Real grade		
		Mean	St. Dev.	p-value	Mean	St. Dev.	p-value
All cases	2D – 3D DTM	0.103	0.250	<0.001	0.105	0.251	<0.001
	2D – 3D DSM	-0.113	0.278	<0.001	-0.111	0.277	<0.001
	3D DTM – 3D DSM	-0.216	0.296	<0.001	-0.215	0.295	<0.001
1) Vegetation	2D – 3D DTM	0.157	0.236	<0.001	0.159	0.239	<0.001
	2D – 3D DSM	-0.200	0.273	<0.001	-0.197	0.274	<0.001
	3D DTM – 3D DSM	-0.356	0.319	<0.001	-0.355	0.318	<0.001
2) Cut side	2D – 3D DTM	-0.019	0.209	0.263	-0.016	0.204	0.344
	2D – 3D DSM	-0.051	0.212	0.004	-0.049	0.208	0.005
	3D DTM – 3D DSM	0.032	0.059	<0.001	-0.033	0.061	<0.001
3) Crest	2D – 3D DTM	-0.045	0.192	0.009	-0.042	0.189	0.012
	2D – 3D DSM	-0.045	0.192	0.009	-0.042	0.189	0.012
	3D DTM – 3D DSM	0.000	0.000	1*	0.000	0.000	1*
4) Clear	2D – 3D DTM	0.287	0.277	<0.001	0.287	0.279	<0.001
	2D – 3D DSM	0.230	0.256	<0.001	0.228	0.256	<0.001
	3D DTM – 3D DSM	-0.058	0.130	0.001	-0.059	0.137	0.001

* The correlation coefficient since the identical samples cannot be compared by the t-test.

Overall, the results show that there is a significant difference in the estimated P_{nc} values between the three ASD modelling methods, both when flat profile and the real grade were contemplated. Only the difference in the pair (2D – 3D DTM) in the second group resulted non-significant. Therefore, this emphasizes the significant implications of using different ASD modelling methods in evaluating the risk associated with a specific design. It can also be noted that the models that overestimated the ASD now underestimate the P_{nc} . The standard deviation values are as high as 0.318, whether they correspond to flat grade or actual grade, meaning that the difference in the evaluated risk is significant. Regarding the grade classification, the results under the flat-profile assumption are similar to its counterpart under the real-grade assumption.

For the first group (i.e. curves with vegetation by the roadside), the estimated P_{nc} mean values indicate that the 3D DTM underestimates the P_{nc} . The comparison (2D - 3D DSM) shows that a

higher risk resulted in the 3D DSM ASD model. This can be expected from the ASD results comparison.

As occurred for the ASD values, P_{nc} values in the third group (crest), pair (3D DTM – 3D DSM) could not be compared by the t -test since they resulted identical. This arises because all the inputs of the corresponding LSF were equal: the position at which the minimum ASD on the curve were produced coincide, therefore the speed distribution at this position as well as the tangential friction distribution are equal for both models. The correlation coefficient is displayed instead. The other two pairs involving the 2D modelling present very small P_{nc} differences, but owing to the small variance the null hypothesis is also rejected, i.e. the difference is significant at the 95 % confidence interval.

The last group (i.e. clear highway segments) shows the greatest P_{nc} differences, especially in those pairs in which the 2D ASD is involved. The 2D ASD method overestimates the risk of failure by more than a 20 % on average. As in most cases in the comparison of P_{nc} values, the corresponding shorter ASD values yield higher P_{nc} values.

Another noteworthy finding is that the significance of the differences in the P_{nc} values is generally higher than the significance of the differences in the estimated ASD values. This is reasonable as when the null hypothesis was rejected for the ASD values, even more so if there are other differences that came from diverse values of both speed and tangential friction factor as inputs in the LSF. For example, the compared pairs associated with non-significant differences in the estimated ASD values (e.g. the 2D–3D DSM pair when comparing the whole cases), have significant differences in the estimated P_{nc} values at 95% confidence level. This can be explained by the non-linear effect of the estimated ASD value on the value of P_{nc} estimated through the reliability analysis. This emphasizes the importance of considering the 3D modelled sight distance, particularly the 3D DSM ASD, when evaluating the associated risk either in highway design or during the service life. Thus, the results indicate that the safety implications of the ASD modelling approach can be significant.

Figure 5.15 shows the empirical cumulative distribution plot of the P_{nc} of all ASD modelling results. The pairs of curves that resulted when flat profile and real grade were considered for each ASD estimation approach are very similar. By comparing the cumulative distribution curves, it can be observed that small P_{nc} values which are close to zero are less frequent in the 3D ASD DSM comparing to the other two approaches. Moreover, as the cumulative frequency grows, the 3D DSM yields higher P_{nc} values than the other two approaches. On the contrary, the 3D DTM approach accumulates the lowest P_{nc} values. Only at the highest P_{nc} values (approximately over 0.84), the 3D DTM curve exceeds the 2D one. Also, the empirical cumulative frequency plots show a frequent incidence of high P_{nc} values. That is, most of the analyzed cases showed to entail a high risk so that their current design does not meet the standards.

The joint frequencies for relevant P_{nc} values in design are exhibited in Table 5.7. These values correspond to the empirical cumulative frequency curves in Figure 5.15. The values in Table 5.7 represent the share of cases whose P_{nc} are below a designated relevant value for each ASD model. The results give evidence that, in this range of values, the 3D DTM approach underestimates the P_{nc} when compared to the other two approaches. The 3D DSM, on the other hand, yields the lowest number of cases complying with the designated relevant thresholds.

Figure 5.15. Empirical cumulative distributions of P_{nc} of the whole curve sample.Table 5.7. Comparison of empirical cumulative frequencies at design P_{nc} values.

P_{nc}	2D	3D DTM	3D DSM
0.001	0.0570	0.2258	0.0498
0.002	0.0702	0.2531	0.0573
0.005	0.0875	0.2959	0.0911
0.01	0.1251	0.3462	0.1137
0.02	0.1563	0.3918	0.1393
0.05	0.2227	0.4736	0.1900
0.10	0.3002	0.5393	0.2496
0.15	0.3695	0.5779	0.2938
0.20	0.4241	0.6314	0.3429
0.50	0.6755	0.7693	0.5482

Figure 5.16 discloses the empirical cumulative distribution plots according to the assumed design speeds. The same pattern of risk overestimation and underestimation as in the general graph (Figure 5.15) is followed in all of them. Low P_{nc} are less frequent in the analysis using the 2D and the 3D DSM approaches in all cases. The adherence among the three curves is low in general. Particularly on highways where design speed is 100 km/h, the maximum P_{nc} values are lower than in the others. In this case, the 3D DSM approach produces significantly more frequent P_{nc} values above 0.4 than the other two approaches. In the case of off-standard roads, the 3D DSM approach produces very different P_{nc} values in all ranges.

Figure 5.16. Empirical cumulative distributions of P_{nc} according to design speeds: a) $v_d = 100$ km/h; b) $v_d = 80$ km/h; c) $v_d = 60$ km/h; d) $v_d = 40$ km/h; e) off-standard.

Theoretically, the area below the curve in the graph could be interpreted as an estimate of the overall facility level of safety. Moreover, sets of P_{nc} values result more consistent as the cumulative probability curve turns nearly parallel to the vertical axle. In such a case, the P_{nc} values would range a shorter interval, the design being more homogeneous. This can be quantified by the standard deviation of the P_{nc} values. Both sets of values are shown in Table 5.8. In this regard, Figure 5.16 shows that the theoretical level of safety decreases as the design speed decreases, except for the case of design speed 60 km/h, where the level of safety is the lowest one. Concerning the consistency of values, more inconsistent levels are found as the design speed decreases. The lesser consistent values are found in the curves that correspond to design speeds 60 and 40 km/h, depending on the ASD modelling technique.

Table 5.8. Theoretical safety level and consistency of P_{nc} values.

v_d (km/h)		2D		3D DTM		3D DSM	
		Flat	Grade	Flat	Grade	Flat	Grade
100	Safety level	0.740	0.734	0.837	0.833	0.736	0.733
	SD	0.159	0.165	0.195	0.205	0.249	0.255
80	Safety level	0.635	0.631	0.825	0.822	0.705	0.701
	SD	0.302	0.307	0.262	0.264	0.312	0.312
60	Safety level	0.439	0.437	0.590	0.592	0.404	0.400
	SD	0.323	0.325	0.382	0.381	0.355	0.354
40	Safety level	0.643	0.639	0.721	0.719	0.579	0.579
	SD	0.333	0.337	0.360	0.361	0.378	0.376
Off-std	Safety level	0.742	0.738	0.814	0.812	0.500	0.501
	SD	0.260	0.268	0.285	0.288	0.374	0.260

5.3.6.3. Scatter plots

Figure 5.17 illustrates the influence of the estimated ASD value on the reliability analysis results. The six series show, approximately, the same pattern. The curves where the ASD was higher than 500 m, have very small P_{nc} values (close to zero). In contrast, cases with ASD below 100 m have P_{nc} values ranging from 0 to 1. The most visible difference is that the 2D approach did not yield cases with very low ASD and high P_{nc} values. On the right part of the graph, a defined boundary exists between the area with points and the area without points.

The relation between 50-th percentile operating speeds and P_{nc} values is depicted in Figure 5.18. In this case, the correspondence is weaker than that between the ASD and the P_{nc} . Below 50 km/h, the density of sites with relatively high probability descends whereas the sites with very low probabilities increase, especially those corresponding to the 2D ASD modelling (black markers). This fact would be explained by the square term of speed in the LSF, making the failure more likely as the speed increases. Moreover, the P_{nc} values that correspond to 2D ASD modelling are mainly concentrated in a triangle in the bottom right area. Sets of values aligned in vertical clusters are also noticeable, which correspond to the mean desired speeds at the sites where the station of minimum ASD on the operating speed profile so indicates. In addition, above 100 km/h, the number of sites with high P_{nc} values falls, probably due to the more generous geometric design of highways whose desired speed on tangents exceeds 100 km/h. Be

that as it may, Figure 5.17 exhibits a clearer pattern than Figure 5.18, indicating the prevalence of the effect of the ASD over that of the speed in SSD analysis.

Figure 5.17. Scatter plot of ASD and P_{nc} values.

Figure 5.18. Scatter plot of the initial speed and P_{nc} values.

5.3.7. Effect of grade

To investigate the effect of grade on the P_{nc} values, the whole sample of 402 curves on both directions and the three ASD estimation methods were considered, resulting in 2,412 cases. As exhibited in Table 3.1, the sample includes highway links located on rolling, hilly and mountainous terrains. The sample comprises an ample variety of grades ranging approximately from -0.1 to 0.1 (Figure 5.19). The Pearson correlation coefficient provides a first insight of the effect of grade on the P_{nc} . When comparing the grade and the P_{nc} variation between the sample that assumed flat profile and the one with the actual grade, the correlation coefficient was -0.648 for the 2D ASD modelling, -0.527 for the 3D DTM ASD modelling and -0.640 for the 3D DSM ASD modelling. The results are consistent since a negative grade (downhill) increases the P_{nc} and the positive grade reduces it. It also makes sense that the correlations are higher in absolute value at the 2D ASD and 3D DSM sets since the ASD values are shorter and the influence of grade is likely to be higher.

Figure 5.19. Histogram of grade frequencies.

Next, the 2,412 values were put all together in a single sample to evaluate the effect of grade more in detail. The 3D scatter plot in Figure 5.20 represents the cloud of points that resulted from depicting the P_{nc} values under the flat-profile assumption on the x axis, the corresponding grades on the y axis, and the P_{nc} variation when incorporating the grade in the reliability analysis on the vertical axis. The points are displayed in darker grey if the difference is negative and lighter grey if difference is positive. The effect of grade is more evident at the middle values of P_{nc} . With the aim of illustrating, an interpolation surface was computed to better describe the effect of grade. For this task, the domain was divided into a mesh so that the interpolated surface would represent adequately the effect on every region of the domain. The mean value of the P_{nc} variation was calculated at each cell. These values were the input of a multiple linear

regression where the input features are polynomial terms up to grade 3. The shape of the surface shows that the effect of grade is negligible at the spots where the P_{nc} is close to 0 or close to 1, which is a consistent result. Indeed, the very low chances of striking an obstacle, where the ASD is more than sufficient, cannot be counteracted by uphill grade or downhill grade. The same applies to sites with extremely high risk. Conversely, the cases of intermediate P_{nc} values are significantly affected by the existing grade.

Figure 5.20. Cloud of points and interpolated surface using regression.

The contour graph derived from the surface in Figure 5.20 is depicted in Figure 5.21. It shows that the positive effect of uphill grades on safety is slightly lower than the counterpart effect of downhill grades.

5.4. Safety performance functions incorporating reliability-based risk results

5.4.1. Statistical model

In this part, a safety performance function relating the P_{nc} (reliability measure) to total collisions was developed. The data set comprises collision and traffic volume data for the selected two-lane rural highways in the Region of Madrid.

Figure 5.21. Contour plot of the effect of grade.

To address the problem of over-dispersion that usually exists in traffic collisions datasets, it is typically assumed that the number of collisions (in this case over a five-year period) Y behaves as a negative binomial random variable. Hence, a GLM was adjusted with mean and variance:

$$E(Y) = \mu \quad \text{Var}(Y) = \mu + \frac{\mu^2}{\kappa} \quad (5.28)$$

For road segments, the mean μ is related to various site-specific traffic, geometric and environmental variables using the following model form:

$$\mu = a_0 \cdot AADT^{a_1} \cdot L \cdot \exp\left(\sum_{j=1}^m b_j \cdot x_j\right) \quad (5.29)$$

This model can be converted into a linear form by means of the following link function:

$$\log(\mu) = \log(a_0) + a_1 \cdot \log(AADT) + \log(L) + \sum_{j=1}^m b_j \cdot x_j \quad (5.30)$$

where:

- μ = expected collision frequency in 5 years;
- L = segment length, m;

$AADT$ = annual average daily traffic, veh. /day;
 a_0, a_1, b_j = model parameters;
 x_j = model variables

Accident records in Spain are managed by the Administration through the Directorate-General of Traffic (*Dirección General de Tráfico* [DGT]). Only accidents with casualties (injured and fatalities [I+F]) were contemplated since the reliance of property-damage-only (PDO) accident data is very low. Attributes in records include the highway code, kilometer (km) and hectometer (hm).

Besides the assumptions for site selection described in section 5.3, curves at or near intersections and driveways were removed from the sample to avoid undesirable additional effects. Also, the accident records were examined to remove inconsistent data, i.e. attributes indicating features not corresponding to the km and hm provided. In addition, to adequately assign accident locations to the curves, the minimum curve length considered including spirals was 80 m. The identification of km and hm could easily be performed on GIS, them being linearly interpolated on the centerline between the milestone (hm = 0) locations.

5.4.2. Results and discussion

The P_{nc} results of 213 curves, considering the grade in profile and the 3D ASD using the DSM, were used to adjust the generalized linear model. The parameters of the adjusted model are shown in Table 5.9. The regression coefficient of the logarithms of traffic volume is less than 1, significant at the 95% confidence interval and positive (as expected) indicating a positive relationship between predicted collisions and AADT. The coefficient of the P_{nc} is positive and the p-value is significant at the 95% confidence interval. The percentage of deviance of the dependent variable explained by the model equals 35.3%. The overdispersion value $1/\kappa$ obtained justifies the use of the negative binomial distribution.

Table 5.9. Coefficients and p-values of the negative binomial model.

Variable	Coefficient	p-value
Intercept	exp(-12.0894)	<0.001
AADT	0.6426	0.003
P_{nc}	0.5353	0.022
$1/\kappa$	2.7764	-

A rule of thumb for a well-fitted model and a relatively large number of observations indicates that the expected value of the Pearson χ^2 should be approximately equal to the number of degrees of freedom. In this model, the Pearson χ^2 is 216.0875 whereas the degrees of freedom equal 210. Table 5.10 shows the estimated correlations between the coefficients in the fitted model. These correlations can be used to detect the presence of serious multicollinearity, i.e., correlation amongst the predictor variables. In this case, there is no correlation with absolute value greater than 0.6.

Finally, Figure 5.22 shows the relationship between predicted collisions and P_{nc} (solid line) with 95% confidence limits (dashed lines), for a road segment with average exposure (length = 152.5

m and AADT = 2,646 veh/day). As it can be expected, the curves show an increase of predicted collisions with P_{nc} , them being all above zero.

Table 5.10. Correlation matrix for coefficient estimates.

Intercept			
-0.4937	AADT		
-0.5119	0.5610	Curve length	
-0.2542	-0.0953	0.0128	P_{nc}

Figure 5.22. Relationship between predicted collisions and the P_{nc} with 95% confidence limits.

5.5. Concluding remarks

Despite the stochastic nature of events, highway design guidelines broach them in a deterministic manner. In this chapter, SSD has been studied using two different methods, both based on a probabilistic approach. The framework is provided by reliability theory. The first method utilizes the Monte Carlo sampling technique, and the second one exploits a more complex numerical method (FORM). Each method enables the research to focus on different variables of the problem. The sampling method provides a flexible structure to accommodate conditioned probability at ease as well as additional calculations not necessarily related to stochastic variables if programmed from scratch. The FORM requires noticeably more complex algorithms that increase the computational performance, at the expense of flexibility.

Some results delivered in the first method support conventional assumptions made in sight distance studies, for example the lower risk of non-compliance entailed by heavy vehicles, most likely owing to both their lower speed values at which they normally travel and the longer ASD provided by the highest position of the observer. In further developments, the speed at which

the hypothetical collision is produced or the margin of non-compliance could be deemed as a measure of potential consequences for a complete estimation of accident risk.

In connection with the foregoing, reliability analysis provides a surrogate tool for highway safety performance analysis. Thus proactive measures could be taken to address safety issues without relying on accident records. In particular, a step forward towards the development of CMFs for sight restrictions is taken through this study.

In the study using the FORM, the assumptions made to create the operating speed model and a complementary 2D sight distance estimation method are first presented. Whereas the differences between 2D and 3D sight distance modelling outcome were already well known, the potential impact of such differences on the SSD remained undisclosed. It was observed that a non-linear effect is produced when the reliability results (P_{nc}) are examined. Indeed, apparently non-significant differences in the ASD values may lead to significant differences in the probability of collision. This fact backs the use of DSMs for sight distance modelling as they comprise sight restrictions adequately and thus deliver the most realistic outcome also in terms of safety.

When the curve sample was classified according to the design speeds of the highways, the results showed that the cumulated probabilities of the P_{nc} values were lower and more consistent as the design speed increased. Hypothetically, the area below the curve in the cumulative probability graph could be interpreted as an estimate of the overall facility level of safety. Moreover, the cumulative probability curves can be considered more consistent as the range of values is smaller (Figure 5.23).

Figure 5.23. Consistent and non-consistent P_{nc} cumulative probability curves.

Finally, a SPF was adjusted incorporating reliability measures, namely the P_{nc} values. The most common exposure variables (AADT and segment length) as well as the P_{nc} resulted significantly greater than 0 at the 95 % confidence interval.

Chapter 6

THREE-DIMENSIONAL ALIGNMENT COORDINATION

6.1. Introduction

Whereas highway geometric design is performed separately in plan, profile and cross section, the driver perceives the spatial result of their superposition. Thus highway design engineers must be aware of the importance of an appropriate alignment coordination. This means that an adequate fitting between the horizontal and the vertical alignment projections must be achieved. The alignment must flow in a predictable manner from the driver's view providing a positive guidance when travelling at high speed. Inappropriate spatial alignment coordination leads to visual discontinuities that may violate drivers' expectations, jeopardizing traffic operation and, therefore, safety.

The study of sight distance under a 3D approach facilitates the analysis of 3D alignment coordination. In particular, concealed horizontal curves and sight-hidden dips can be identified and quantitatively described at ease. In the previous chapter, 3D alignment coordination was already addressed to some extent as the ASD on horizontal curves was studied contemplating the 3D nature of the roadway and roadsides, and its effect on safety was studied by means of reliability theory.

Interim hidden sections are usually linked to a more generous design in plan than in profile or vice versa. In this chapter, on the one hand, the effect of elevation modelling on the identification and characterization of hidden dips is studied. Also, the layout of alignments in both projections is analyzed to find out the alignment features that affect the most the existence and characterization of these perspective shortcomings. On the other hand, the conveniences and difficulties inherent to the implementation of SSE in motorway design are discussed by means of a case study. Also, a quantitative methodology to avoid hazardous hidden beginning of curves on motorways is proposed.

6.2. Sight-hidden dips on two-lane highways

Sight-hidden dips could be hazardous sections for drivers willing to pass the vehicle that precedes them. In this regard, designers demand guidelines to avoid the appearance of sight-hidden dips in highway alignment, to be able to remove them or, at least, reduce their potential effects on safety. However, all redesign possibilities are not always feasible as right-of-way restrictions must be preserved.

As the presence of roadside items affected the ASD results in the sight distance evaluation, the characterization of sight-hidden dips, especially on in-service highways, may also be inaccurate if the elements affecting sight distance are not incorporated in the elevation model used.

As mentioned in the literature review, Easa (1994a) formulated an analytical model to quantify sight-hidden dips' parameters in 2D. On a sequence grade – crest – grade – sag – grade, and a rather straight overlapped horizontal alignment, the equations delivered provided exact quantitative relations between the vertical alignment geometrics and the parameters of the hidden dip. Even in such a simple case, the equations arisen resulted somewhat cumbersome.

An analytical approach to broach the study of relations between the alignment features and either the presence of a hidden section or the sight-hidden dip parameters would lead to extremely complex models if the horizontal projection, the profile, and the cross section are to be borne in mind.

6.2.1. Methodology

Sight distance was computed on the selected two-lane rural highways using the GIS add-in *Road Sight Distance*. The inputs were the usual DTM01 and the DSM01. In this case, stations were set spaced 5 m apart on trajectories traced at a distance of 1.5 m from the outer edge of the own lane, in accordance with the Spanish standard (Ministerio de Fomento, 2000). The computational process was launched twice for each direction. In both, the driver's eye height (h_1) was set at 1.1 m above the pavement. Sight distance was evaluated in the first one assuming the target height (h_2) at 0.2 m so that all sections where a visual break of the roadway is produced are being contemplated. In the second launch, the target height was set at 1.1 m to examine hidden dips with potential passing issues. A distinction is therefore made between detected perspective shortcomings and sight-hidden dips that may entail PSD issues.

Crest – sag sequences were sought in the vertical alignment as they are typically the profiles where sight-hidden dips are found, albeit this layout does not lead to hidden dips in all cases. In order to avoid the study of doubtful cases, the grade variation on both vertical curves must exceed 0.01. Contemplating both driving directions, a total of 368 cases were observed that complied with these requirements.

Next, these cases were classified according to what is produced in the perspective recreation using either elevation model in ArcScene, and what is found in the real world aided by the available on-board video recordings and Google Street View. A first classification distinguishes whether a hidden dip is produced. Two additional classes of hidden dips are considered. Diving is a sight-hidden dip where the horizontal alignment is essentially straight (section 2.4.2.1.2) whereas jumping sections are featured by a curved horizontal projection. Jumping sections

might probably deceive drivers less than diving sections because the sight interruption is more evident. For the purpose of the classification, a sight-hidden dip is considered as a diving case if the crest and the sag lie on the same tangent. In this case, vertical alignments are solely responsible for the existence of the hidden dip (Easa, 1994). It is worth stressing that the classification totally matched observed cases between the real world and the DSM01 while it did not with the results using the DTM01.

Table 6.1 shows the number of observations for each class according to the elevation model used in the sight distance evaluation. Hidden interim sections must be necessarily associated with a crest – sag sequence to be computed as a hidden dip. Percentages are referred to the total sample of 368 observations. When the DSM01 is used to study the dataset, no hidden interim section is detected in as many as 70 sections ($q^* > 0.2$ m) that were classified as jumping as per the evaluation on the DTM01. Conversely, the number of diving sections remains almost the same regardless of the elevation model used. In particular, for the evaluation with $h_2 = 0.2$ m, the number of cases remains unchanged whereas only 2 observations out of 45 had hidden interim areas deeper than 1.1 m. According to the criterion used to classify diving sections, the reemerged section cannot be avoided in most cases as the horizontal projection is straight. However, no restriction is specified for the outbound grade regarding its horizontal projection. Therefore, if that grade is overlapped with a horizontal curve with items by the inner roadside such as vegetation, the hidden dip may be enlarged and its parameters increase, including depth of the hidden interim sections.

Table 6.1. Classification of crest – sag sequences on the selected highways.

Elevation model	None	$q^* > 0.2$ m			$q^* > 1.1$ m		
		Diving	Jumping	Total	Diving	Jumping	Total
DTM01	123 (33.4 %)	77 (20.9 %)	168 (45.7 %)	245 (66.6 %)	43 (11.7 %)	115 (31.3 %)	158 (42.9 %)
DSM01	193 (52.4 %)	77 (20.9 %)	98 (26.6 %)	175 (47.6 %)	45 (12.2 %)	48 (13.0 %)	93 (25.3 %)

The dataset of sight-hidden dips is found in Appendix III (digital version only). It is worth noting the large number of hidden dips found in the selected highways, which was one of the objectives pursued in the preliminary visual inspection made on Google Street View (section 3.3.2) for the case study selection.

6.2.2. Parameters of sight-hidden dips

In the literature review, the fundamental parameters that characterize sight-hidden dips were described. Besides the ASD, several authors highlighted the relevance of range, depth, length of hidden interim section, and distance to the end of hidden section (Table 2.3). These parameters can be interpreted as the risk probabilities which jeopardize the safe operation of such sections. The hidden dip range represents the section travelled where the driver willing to pass could wrongly make the decision of initiating the maneuver as they cannot notice if the opposite lane is clear. The length of hidden interim section (deeper than a certain threshold) represents the chances that oncoming traffic might be present on the opposite lane. Finally, the distance to the

end of hidden section could be interpreted as the odds that the driver notices that there is an interim hidden section that may lead to passing issues.

6.2.3. Effect of sight distance modelling inputs on sight-hidden dip characterization

As detailed above, the sight distance computation was performed on both the DTM01 and DSM01 for the study of sight-hidden dips. In each case observed, the corresponding hidden dip parameter values are deduced from the results with either elevation model. In this subsection, the results of a paired comparison of these sight-hidden dips parameter values are presented.

Besides the parameters described in 6.2.2, the sight distance graph and the capabilities of *Road Sight Distance* software permit the definition of an additional parameter, namely the overall area of the interim hidden sections in the sight distance graph. This additional parameter can also illustrate the risk probability of these sections. This area corresponds to the zone in the sight distance graph vertically enclosed between the seen areas. In the example of Figure 6.1, it matches the zone shaded in light brown, it being vertically bounded by zones shaded in green. The order of magnitude is approximately the value of the product of range multiplied by the length of the hidden section.

Figure 6.1. Identification of parameters characterizing sight-hidden dips in the sight distance graph.

Overall statistical measures of the observed sight-hidden dips are shown in Table 6.2 to

Table 6.5. The variation of the target height aims at understanding the effects of the depth of diving. A reduction of parameter values is observed when comparing the statistical measures of the outcome with target height 0.2 m and those with target height 1.1 m. In fact, it turns out that each parameter value is equal or lower in the latter outcome for every case in the dataset. If the same comparison is made between the outcomes of both elevation models at the same target height assumption, the statistical measures decrease for all the parameters in the DSM01. It is partially due to the fact that the hidden areas are shallower than 1.1 m in some cases. Nonetheless, the effect of the elevation model used is not as straightforward as for the target height since variations do not point in the same direction in all cases. Indeed, the presence of roadside sight obstructions may cause that the reemerged section is seen further so that all the other parameters would probably increase (range, length of longest hidden interim section and depth). On the contrary, sight obstructions may conceal reemerged roadway area completely. Therefore, the initial overview that arises from the statistical measures must be checked.

Table 6.2. Statistical facts of observed range values in hidden dips.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2$ m	$q^* > 1.1$ m	$q^* > 0.2$ m	$q^* > 1.1$ m
Count	175	175	175	175
Average	103.6 m	47.0 m	63.9 m	25.0 m
Median	50 m	15 m	35 m	5 m
Standard deviation	116.5 m	88.2 m	79.7 m	59.4 m
Coefficient of variation	112.5 %	187.8 %	124.7 %	237.3 %
Minimum	5 m	0 m	5 m	0 m
Maximum	585 m	530 m	575 m	485 m

Table 6.3. Statistical facts of the length of interim hidden sections in hidden dips.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2$ m	$q^* > 1.1$ m	$q^* > 0.2$ m	$q^* > 1.1$ m
Count	175	175	175	175
Average	251.0 m	154.7 m	198.5 m	106.3 m
Median	190 m	85 m	145 m	35 m
Standard deviation	188.2 m	187.3 m	165.1	152.8 m
Coefficient of variation	75.0 %	121.1 %	83.2 %	143.8 %
Minimum	20 m	0 m	15 m	0 m
Maximum	865 m	785 m	810 m	665 m

The quantification of the differences between hidden dip parameter values of either elevation model outcome is the most relevant. As occurred in section 4.3.2, the samples to be compared cannot be assumed to be normally distributed because stations were set 5 m apart. Therefore, the sight distance output is not a continuous variable but discrete. Then a non-parametric test for paired samples is performed, namely the Wilcoxon signed rank test. To be on the safe side in the comparison, only real observations were considered. The results of the comparison are shown in

Table 6.6. The parameter values were found to be significantly different at the 95 % confidence level for both target height assumptions in all cases. Besides the Wilcoxon p-value, two measures of the paired differences are provided: the average difference and the RMS. It is recalled that the RMS value provides a more robust measure of difference since it removes the effect of sign.

Table 6.4. Statistical facts of the distance to the reemerged roadway in hidden dips.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2$ m	$q^* > 1.1$ m	$q^* > 0.2$ m	$q^* > 1.1$ m
Count	175	175	175	175
Average	409.8 m	277.3 m	334.4 m	201.1 m
Median	306 m	245 m	290 m	150 m
Standard deviation	216.9 m	274.7 m	185.0 m	233.5 m
Coefficient of variation	52.9 %	99.1 %	55.3 %	116.1 %
Minimum	105 m	0 m	65 m	0 m
Maximum	995 m	965 m	975 m	930 m

Table 6.5. Statistical facts of area in graph of the interim hidden sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2$ m	$q^* > 1.1$ m	$q^* > 0.2$ m	$q^* > 1.1$ m
Count	175	175	175	175
Average	21,002 m ²	8,799 m ²	11,422 m ²	3,835 m ²
Median	6950 m ²	1,175 m ²	3,200 m ²	300 m ²
Standard deviation	47204.8 m ²	19,656 m ²	38,549 m ²	10,618 m ²
Coefficient of variation	224.7 %	223.4 %	337.5 %	276.9 %
Minimum	150 m ²	0 m ²	75 m ²	0 m ²
Maximum	474,025 m ²	133,800 m ²	471,600 m ²	76,450 m ²

Table 6.6. Results of the paired samples comparison of hidden dip parameters.

Parameter	Statistical measure	$q^* > 0.2$ m	$q^* > 1.1$ m
Range	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	39.7 m	21.9 m
	RMS	98.2 m	69.4 m
Longest length of interim hidden section	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	52.5 m	48.4 m
	RMS	123.0 m	136.3 m
Distance to reemerged roadway	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	75.4 m	76.3 m
	RMS	154.2 m	211.2 m
Area of hidden sections in graph	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	9,580 m ²	4,964 m ²
	RMS	28,766 m ²	17,053 m ²

Next, the sample is segmented to evaluate the differences in the hidden dip parameter values in the two hidden dip classes separately: diving and jumping sections. First, the parameters of diving sections are analyzed. Table 6.7 to Table 6.10 display statistical measures of the parameters of diving sections. In this group, the differences of the statistical measures between the outcomes under the two target height assumptions are still evident whereas the differences between the outcomes with the two elevation models are not clear for all the parameters.

Table 6.7. Statistical facts of observed range values in diving sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	77	77	77	77
Average	100.5 m	39.4 m	84.2 m	34.6 m
Median	50 m	10 m	45 m	10 m
Standard deviation	111.1 m	81.6 m	96.1 m	76.3 m
Coefficient of variation	110.5 %	206.9 %	114.1 %	220.5 %
Minimum	5 m	0 m	5 m	0 m
Maximum	575 m	485 m	575 m	485 m

Table 6.8. Statistical facts of the length of interim hidden sections in diving sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	77	77	77	77
Average	261.4 m	151.8 m	241.2 m	129.3 m
Median	190 m	75 m	190 m	50 m
Standard deviation	191.0 m	195.9 m	165.5 m	159.7 m
Coefficient of variation	73.0 %	129.1 %	68.6 %	123.5 %
Minimum	30 m	0 m	30m	0 m
Maximum	865 m	785 m	810 m	665 m

Table 6.9. Statistical facts of the distance to the reemerged roadway in diving sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	77	77	77	77
Average	411.2 m	262.2 m	380.3 m	240.8 m
Median	380 m	210 m	355 m	210 m
Standard deviation	207.6 m	278.0 m	186.6 m	247.3 m
Coefficient of variation	50.5 %	106.0 %	49.1 %	102.7 %
Minimum	105 m	0 m	105 m	0 m
Maximum	980 m	960 m	975 m	900 m

After the initial overview, the differences in the parameter values on diving sections were statistically studied. The differences were significant at the 95 % confidence level only at the shallower target height, and the corresponding p-values, although lower than 0.05, they are not as small as for the complete sample (

Table 6.11). For the target height 1.1 m, the hypothesis that the samples come from different distributions could not be rejected in two parameters: the lengths of hidden sections and the distance to reemerged roadway. Furthermore, the average differences and the RMS values are noticeably reduced if compared to those of the complete sample.

Table 6.10. Statistical facts of area in graph of the interim hidden diving sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	77	77	77	77
Average	22,205 m ²	7,498 m ²	18,810 m ²	5,736 m ²
Median	10,175 m ²	625 m ²	5,400 m ²	625 m ²
Standard deviation	57,701 m ²	16,040 m ²	56,535 m ²	14,477 m ²
Coefficient of variation	259.9 %	213.9 %	300.5 %	252.4 %
Minimum	350m ²	0 m ²	300 m ²	0 m ²
Maximum	474,025 m ²	76,450 m ²	471,600 m ²	76,450 m ²

Table 6.11. Results of the paired samples comparison of diving section parameters.

Parameter	Statistical measure	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Range	Wilcoxon p-value	< 0.001	0.020
	Average difference	16.4 m	4.8 m
	RMS	54.8 m	21.0 m
Longest length of interim hidden section	Wilcoxon p-value	0.016	0.052
	Average difference	20.3 m	22.5 m
	RMS	78.3 m	105.2 m
Distance to reemerged roadway	Wilcoxon p-value	< 0.001	0.127
	Average difference	30.8 m	21.4 m
	RMS	83.7 m	134.5 m
Area of hidden sections in graph	Wilcoxon p-value	0.001	< 0.001
	Average difference	1,762 m ²	3,395 m ²
	RMS	5,483 m ²	10,396 m ²

The results of the comparison in diving sections can be further explained by means of the following example. Figure 6.2 depicts a hidden dip produced on a straight section. The corresponding sight-distance graphs are exhibited in Figure 6.3. The range is slightly shorter in the case of the DSM01 output. It begins on station 1+775 for the DTM01 and on station 1+780 for the DSM01. As a result, the longest hidden section corresponds to a different line of sight. The dark brown area represents the hidden sections that resulted from the calculation with target height 1.1 m, which obviously were not seen with target height at 0.2 m either. The shape of the common hidden area remains the same. The evaluation of these lines of sight is conditioned by the roadway.

Figure 6.2. Diving section on highway M-611.

Figure 6.3. Sight distance graphs displaying results of diving section on highway M-611 for a) DTM01 and b) DSM01.

Finally, the parameters of jumping sections are analyzed. Table 6.12 to Table 6.15 exhibit statistical measures of the parameters of jumping sections. The same general patterns as for the whole sample are noticed, i.e. the hidden dip parameter values visibly decrease when the target height rises and when the surface model is utilized.

The significance of the differences was likewise checked on jumping sections. The differences are significant at the 95 % confidence level once again, the null hypothesis that the parameter sets come from the same distribution being rejected in all cases. The average difference and the RMS values even increase slightly if compared to those of the complete dataset.

Table 6.12. Statistical facts of observed range values in jumping sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	98	98	98	98
Average	106.0 m	52.9 m	48.0 m	17.5 m
Median	50 m	15 m	25 m	0 m
Standard deviation	121.1 m	93.1 m	59.9 m	40.4 m
Coefficient of variation	114.3 %	176.0 %	124.7 %	231.2 %
Minimum	5 m	0 m	5 m	0 m
Maximum	585 m	530 m	330 m	305 m

Table 6.13. Statistical facts of the length of interim hidden sections in jumping sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	98	98	98	98
Average	242.8 m	157.0 m	164.9 m	88.3 m
Median	195 m	100 m	115 m	0 m
Standard deviation	186.5 m	181.3 m	157.7 m	145.5 m
Coefficient of variation	76.8 %	115.5 %	95.6 %	164.8 m
Minimum	20 m	0 m	15 m	0 m
Maximum	785 m	665 m	755 m	660 m

Table 6.14. Statistical facts of the distance to the reemerged roadway in jumping sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	98	98	98	98
Average	408.7 m	289.2 m	298.4 m	169.8 m
Median	345 m	270 m	245 m	0 m
Standard deviation	225.0 m	273.0 m	176.5 m	218.3 m
Coefficient of variation	55.1 %	94.4 %	59.1 %	128.5 m
Minimum	105 m	0 m	65 m	0 m
Maximum	995 m	965 m	935 m	930 m

Table 6.15. Statistical facts of area in graph of the interim hidden jumping sections.

Statistical measure	DTM01		DSM01	
	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Count	98	98	98	98
Average	20,057 m ²	9,821 m ²	5,618 m ²	2,341 m ²
Median	5,725 m ²	1,425 m ²	1,790 m ²	0 m ²
Standard deviation	37,235 m ²	22,115 m ²	9,167 m ²	5,734 m ²
Coefficient of variation	185.6 %	225.2 %	163.1 %	244.9 %
Minimum	150 m ²	0 m ²	75 m ²	0 m ²
Maximum	191,150 m ²	133,800 m ²	51,725 m ²	30,875 m ²

Table 6.16. Results of the paired samples comparison of jumping section parameters

Parameter	Statistical measure	$q^* > 0.2 \text{ m}$	$q^* > 1.1 \text{ m}$
Range	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	58.0 m	35.4 m
	RMS	121.9 m	90.9 m
Longest length of interim hidden section	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	77.8 m	68.7 m
	RMS	149.0 m	156.4 m
Distance to reemerged roadway	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	110.4 m	119.4 m
	RMS	192.2 m	255.9 m
Area of hidden sections in graph	Wilcoxon p-value	< 0.001	< 0.001
	Average difference	14,440 m ²	7,480 m ²
	RMS	37,320 m ²	22,264 m ²

An example of a hidden dip with jumping reemerged area is shown in Figure 6.4. In this case, the sag and the outbound grade are overlapped with a horizontal curve.

Figure 6.4. Hidden dip with jumping reemerged section in highway M-222.

Figure 6.5a displays the graph resulting from the sight distance evaluation on the DTM01 while Figure 6.5b shows the identical section evaluated on the DSM01. It can be noticed that the range is significantly reduced on the DSM01 and, therefore, the other parameters: the length of interim hidden section, the distance to the reemerged roadway area and the area of interim hidden sections in graph. In this example, the shape of the common hidden area also remains the same (stations 4+215 to 4+245), it corresponding to lines of sight conditioned by the roadway. The outcome differences are mainly caused by the sight obstruction by the inner roadside of the horizontal curve to the left. The reemerged section is partially concealed so the driver is not able to see it up to station 4+215.

Figure 6.5. Sight distance graphs depicting results of jumping section on highway M-222 for a) DTM01 and b) DSM01.

6.2.4. Effect of alignment parameters on sight-hidden dips

6.2.4.1. Methodology

In this subsection, the effect of alignments on the existence of sight-hidden dips, as well as their parameters, is researched. Several statistical models will be adjusted to provide insight into the issue. These models comprise limited variable dependent models (logit) for binary variables (existence of hidden dip) and linear regression for continuous variables. As in the previous subsection, the case study comprises the dataset of 386 observations found on the selected highways where a crest – sag sequence is located. The assumptions made in the previous subsection are kept. Four sets of results come out of the two elevation models (DTM01 or DSM01) and the target heights assumed (h_2 equal 0.2 or 1.1 m). As already mentioned in previous subsections of this chapter, the target height was also set at 1.1 m to account for sections with potential passing issues.

The dependent variables considered are exhibited in Table 6.17. The two variables above are binary variables and refer to the existence and the type of hidden dip. Moreover, the variable *Jump* is contemplated only when the dataset comprises observations where a sight-hidden dip is detected. The other four variables are the hidden dip parameters as described in the previous subsection. It is worth recalling that these variables will take four different values for each of the four possible combinations of elevation models and target height.

Table 6.17. Dependent variables considered on crest – sag sequences.

ID	Name	Type	Description	Unit
1	<i>SHD</i>	Binary	Existence of hidden dip	-
2	<i>Jump</i>	Binary	Diving (0) / jumping (1)	-
3	<i>Range</i>	Continuous	Range where hidden section is perceived	m
4	<i>L_{HS}</i>	Continuous	Maximum length of interim hidden section	m
5	<i>D_{EHS}</i>	Continuous	Distance to the end of hidden section	m
6	<i>A_{HS}</i>	Continuous	Area of hidden sections in sight distance graph	m

The basic independent variables are described in Table 6.18. They refer to measures of alignments on the horizontal and the vertical projection as well as a cross section parameter, namely the roadway width. The parameters of the vertical alignments are shown in Figure 6.6. Variables from rows 2 to 12 refer to bare vertical alignment measures, whereas in rows 13 to 25 relations between alignments on either projection are quantified. The latter ones include the curvature of the radius overlapped with the crest, the angle turned on each vertical alignment and if the alignments meet the definition of SSE of the German standards for rural roads (FGSV, 2012). The angles include two variations: absolute value of the overall deflection angle and sum of the absolute values of the angles turned. A VBA script was devised to compute the deflection angles on each vertical alignment and to check the existence of SSE. From these basic independent variables, more variables were derived. Square root, logarithm, square and inverse values were calculated, for the variables in which it is mathematically correct, to create more variables that may adjust statistical models better. In addition, sums of deflection angles in successive elements and lengths taken in groups of three, four and five were computed to create

more variables. Finally, the product of variables defined for the same elements were calculated as interaction terms. As a result, a total of 487 variables were contemplated in the dataset for modelling the existence of sight-hidden dips and their parameters.

Table 6.18. Description of the basic independent variables on crest – sag sequences.

ID	Name	Description	Unit
1	W_{RW}	Roadway width	m
2	L_{in}	Length of grade before crest curve	-
3	$K_{V,0}$	Curvature parameter of crest curve	m
4	θ_0	Algebraic difference in grades by the crest curve	-
5	L_0	Length of the crest curve	m
6	L_m	Length of the interim grade	m
7	$K_{V,1}$	Curvature parameter of sag curve	m
8	θ_1	Algebraic difference in grades by the sag curve	-
9	L_1	Length of the sag curve	m
10	L_{out}	Length of the grade beyond the sag curve	m
11	D_{crests}	Distance between the middle points of successive crests	m
12	θ_{sags}	Algebraic difference in grades for sequence of sags	-
13	C_{crest}	Inverse of curve radius overlapped with the crest	m ⁻¹
14	Ω_{in}	Deflection angle on grade before crest curve	gon
15	Ω_0	Deflection angle on crest curve	gon
16	Ω_{mid}	Deflection angle on interim grade	gon
17	Ω_1	Deflection angle on sag curve	gon
18	Ω_{out}	Deflection angle on grade beyond sag curve	gon
19	Ω_{crests}	Deflection angle on alignments between successive crests	gon
20	SSE_{in}	The grade before crest curve is SSE	-
21	SSE_0	The crest curve is SSE	-
22	SSE_m	The interim grade is SSE	-
23	SSE_1	The sag curve is SSE	-
24	SSE_{out}	The grade beyond sag curve is SSE	-
25	SSE_{any}	Any of the alignment elements is SSE	-

For the dependent variables *SHD* and *Jump*, logit models were adjusted including every independent variable one by one, without intercept term. This process was repeated four times, incorporating the results whose inputs were each of the two elevation models and each of the two target heights. In such cases, the dataset utilized comprised the 386 observations.

Linear regression models were adjusted for the rest of the dependent variables, incorporating every independent variable also one by one and without intercept term. In these models, the process was also reiterated four times: on the two elevation models available and under the two target height assumptions. The datasets utilized in each case were those where the corresponding hidden dips were observed (total cases with $q^* > 0.2$ m in Table 6.1).

Figure 6.6. Depiction of the vertical alignment parameters considered.

6.2.4.2. Results and discussion

6.2.4.2.1. Effect of alignment variables on the existence of hidden dips

More than 200 adjusted logit models presented features with coefficients significantly different from 0 (p -value < 0.05 , and in many cases below 0.001) for both target heights. However, the goodness of fit was low in any case (less than 0.05). The basic variables whose derived variables had the lowest p -values were those exhibited in Table 6.19. These variables typically showed, although small, the highest goodness of fit values.

Table 6.19. Most significant variables affecting the probability of sight-hidden dips occurring on DTM01.

Rank	$h_2 = 0.2 \text{ m}$		$h_2 = 1.1 \text{ m}$	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	θ_1	Positive	θ_1^{-1}	Negative
2	θ_{sags}	Positive	θ_{sags}^{-1}	Negative
3	W_{RW}	Positive	Ω_m	Negative
4	θ_0	Positive	L_1^{-1}	Negative
5	L_0	Positive	W_{RW}	Negative

It is worth mentioning that, except for the width, the variables in Table 6.18 for $h_2 = 0.2 \text{ m}$ correspond to vertical alignment features, namely grade variation on vertical curves. This indicates that, on the DTM, the parameters that may have a greater effect on the existence of sight-hidden dips are those of the vertical alignment, yet the great variance in data does not lead to draw strong conclusions in this respect. The roadway width appears both as an independent term and as interaction term with differences in grades (θ). The sign of the coefficients were consistent among the derived variables and with what could be expected. The positive sign shows that the logarithm of odds of the existence of a hidden dip increases as these variables increase. In the case of the roadway width, the sign is consistent as well since wider

roadways provide greater clearances that reduce the chances of concealing the reemerged sections, typically occurring in the case of hidden dips with jumping.

For $h_2 = 1.1$ m, the most relevant variables resulted different and more varied than in the models of the previous set. The predicted effect of both algebraic difference in grades are lower, although consistent with that under $h_2 = 0.2$ m (the inverse value with a negative coefficient indicates that it affects in the same way). It is worth noting that the horizontal deflection angle on the interim grade reduces the chances of a hidden dip occurring.

Given the poor goodness-of-fit values, the existence of a sight-hidden dip seems to depend greatly on features that cannot be incorporated in the model (i.e. roadsides) rather than on those of the alignment. Therefore, the alignment redesign by means of the alteration of the variables in Table 6.19 is far from ensuring the removal of the hidden dip.

The counterpart analysis of variables was performed on the DSM01 results dataset. The variables that resulted more significant in the individual logit models are shown in Table 6.20. More than 200 features resulted significantly different from 0 at the 95 % confidence interval under both target heights. At $h_2 = 0.2$ m, the prevalence of horizontal deflection angles as the variables with higher impact is noticeable. Particularly, the sum of the absolute values had better fit than the absolute value of the sums of the deflection angles. The negative sign of the estimates indicate that a curved horizontal projection reduces the probability of a sight-hidden dip created by the crest – sag sequence as can be expected. At $h_2 = 1.1$ m, the results are very similar to those of the DTM01. This means that the variables that affect the depth of diving the most, namely difference in grade at sags and deflection angle on interim grade and on sags, remain almost the same regardless of the elevation model utilized and thus regardless of sight obstructions. In both lists of features, the highest goodness of fit is also achieved in conjunction with the highest statistical confidence levels.

Table 6.20. Most significant variables affecting the probability of sight-hidden dips occurring on DSM01.

Rank	$h_2 = 0.2$ m		$h_2 = 1.1$ m	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	$\Omega_m + \Omega_1$	Negative	θ_1^{-1}	Negative
2	Ω_1	Negative	θ_{sags}^{-1}	Negative
3	$\Omega_{in} + \Omega_0 + \Omega_m$	Negative	Ω_m	Negative
4	$\Omega_0 + \Omega_m + \Omega_1$	Negative	Ω_{sags}	Negative
5	$\Omega_0 + \Omega_m + \Omega_1 + \Omega_{out}$	Negative	L_1^{-1}	Negative

6.2.4.2.2. Effect of alignment variables on the hidden dip range

In this subsection the results of the linear regression models that predicted the effects of alignment variables on the range of hidden dips are presented. The results for the DTM01 are exhibited in Table 6.21. At $h_2 = 0.2$ m, the overall algebraic difference in grades at sags was found to increase the range of hidden sections the most. The roadway width, the algebraic difference in grades at the crest as well as the length of the sag are also the main variables affecting the hidden dip range. It must be recalled that the algebraic difference in grades incorporate sign so,

in absolute value, that of the crest increases the range. Thus the signs of the estimates are coherent with the results expected. The derived variables found to be significant correspond to logarithm and square root values of the basic variables, meaning that the effect is not linear but decreases as the value of the variable increases. For $h_2 = 1.1$ m, the variables appearing to affect most the range remain essentially unchanged, including the estimate signs. In general, the individual goodness-of-fit values are much greater in these models than in the logit ones, the first ones reaching values up to 0.45 ($h_2 = 0.2$ m) and 0.29 ($h_2 = 1.1$ m).

Table 6.21. Most significant variables affecting the range of hidden dips on DTM01.

Rank	$h_2 = 0.2$ m		$h_2 = 1.1$ m	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	θ_{sags}	Positive	θ_{sags}	Positive
2	θ_1	Positive	W_{RW}	Positive
3	W_{RW}	Positive	θ_1	Positive
4	L_1	Positive	$L_{in} + L_0$	Positive
5	θ_0	Negative	L_{in}	Positive

Table 6.22 shows the variables that were found to exert a greater influence on the range of hidden dips on the DSM01. With respect to the DTM01, most variables that affect the most the range are also in the list of the DSM01. In the case of the lower target height, differences in grades are the most relevant basic variables, yet the specific variables that better adjusted were the corresponding logarithms and square roots, indicating that their effect on the range is not linear. On sight-hidden dips deeper than 1.1 m, except for the difference in grades θ_{sags} , they turned out to be more affected by the vertical alignment lengths. The individual goodness-of-fit values reach up to 0.41 ($h_2 = 0.2$ m) and 0.19 ($h_2 = 1.1$ m).

Table 6.22. Most significant variables affecting the range of hidden dips on DSM01.

Rank	$h_2 = 0.2$ m		$h_2 = 1.1$ m	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	θ_0	Negative	L_{in}	Positive
2	θ_{sags}	Positive	θ_{sags}	Positive
3	W_{RW}	Positive	$L_{in} + L_0$	Positive
4	θ_1	Positive	L_0	Positive
5	$K_{V,0}$	Positive	W_{RW}	Positive

6.2.4.2.3. Effect of alignment variables on the length of interim hidden section

In this subsection, the results of the linear regression models that predicted the effects of alignment variables on the length of the interim hidden sections are presented. Those on the DTM01 are exhibited in Table 6.23. At the lowest target height, the basic variables found to be more statistically significant in the individual models are varied whereas the vertical alignment lengths prevail at the highest target height. The signs of the estimates are coherent with what can be expected. For example, the increase in the length of the vertical alignments would favor longer hidden interim sections. The lengths and vertical curvature parameters adjust better if

the square root value is used as input. Algebraic differences in grades and the sag parameter adjust better when their logarithms and square root values are input, indicating their positive effect decreases as they increase. As the values of the difference in grade are smaller than 1 (in absolute value), the increase of its logarithm value produces an expansion of the length of hidden section. Goodness-of-fit values remain low for all the variables ($R^2 < 0.12$). At the higher target height, the most relevant variables are the lengths of the vertical alignments and, in particular, the distance between successive crests. The sign of coefficients are consistent and the goodness of fit is better in this set as individual R^2 values reach up to 0.47.

Table 6.23. Most significant variables affecting the length of interim hidden section on DTM01.

Rank	$h_2 = 0.2 \text{ m}$		$h_2 = 1.1 \text{ m}$	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	$L_1 + L_{out}$	Positive	D_{crests}	Positive
2	$L_m + L_1 + L_{out}$	Positive	$L_1 + L_{out}$	Positive
3	D_{crests}	Positive	$L_m + L_1 + L_{out}$	Positive
4	$K_{V,1}$	Positive	$L_0 + L_m + L_1 + L_{out}$	Positive
5	θ_0	Negative	$L_{in} + L_0 + L_m + L_1 + L_{out}$	Positive

Table 6.24 shows the information about the most relevant variables on the DSM01. For $h_2 = 0.2 \text{ m}$, the basic variables that individually showed the highest goodness of fit coincide with those that were statistically more significant. All of them showed the best fit when the square root values were used as input, the effect thus being non-linear (increase less than proportionally). R^2 was as high as 0.69 in the case of the most relevant variable. The coefficients were positive as expected. For $h_2 = 1.1 \text{ m}$, the basic variables are also related to vertical alignment lengths with coefficients significantly different from 0, but showing lower goodness-of-fit values (up to 0.43). The two first features showed better fit with linear adjustment while the other three adjusted better in the shape of the square root.

Table 6.24. Most significant variables affecting the length of interim hidden section on DSM01.

Rank	$h_2 = 0.2 \text{ m}$		$h_2 = 1.1 \text{ m}$	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	$L_{in} + L_0 + L_m + L_1 + L_{out}$	Positive	$L_0 + L_m + L_1$	Positive
2	D_{crests}	Positive	$L_{in} + L_0 + L_m + L_1 + L_{out}$	Positive
3	$L_m + L_1 + L_{out}$	Positive	D_{crests}	Positive
4	$L_0 + L_m + L_1$	Positive	$L_{in} + L_0 + L_m$	Positive
5	$K_{V,1}$	Positive	L_m	Positive

6.2.4.2.4. Effect of alignment variables on the distance to reemerged roadway

In this subsection the results of the linear regression models that predicted the effects of alignment variables on the distance to reemerged roadway are presented. The results on the DTM01 are exhibited in Table 6.25. As occurred for the length of the longest hidden section, the variables that explained the dependent variable the most satisfactorily are the sums of lengths of the vertical alignments (input as square root), followed by the vertical curve parameters

(input as logarithm). The latter ones showed better adjustment if input as logarithm. The patterns shown under both target height assumptions are very similar. The goodness-of-fit values on the individual regression equations reach up to 0.82 ($h_2 = 0.2$ m) and 0.6 ($h_2 = 1.1$ m), and the coefficients are positive as expected.

Table 6.25. Most significant variables affecting the distance to reemerged roadway on DTM01.

Rank	$h_2 = 0.2$ m		$h_2 = 1.1$ m	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	$L_0+L_m+L_1$	Positive	L_m+L_1	Positive
2	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive	$L_0+L_m+L_1$	Positive
3	D_{crests}	Positive	$L_{in}+L_0+L_m+L_1$	Positive
4	$K_{V,1}$	Positive	D_{crests}	Positive
5	$K_{V,0}$	Positive	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive

The results on the counterpart analysis on the DSM01 are displayed in Table 6.26. In the set that corresponds to $h_2 = 0.2$ m, the most relevant variables are varied, the element lengths being the fundamental ones. The square root (lengths only) and logarithm values of these basic variables delivered the models with better fit, indicating a non-linear effect. Goodness-of fit values reached up to 0.83. In the set of $h_2 = 1.1$ m, element length are the dominant variables, their square root values providing the best fit. In the best case, the share of the model that was explained by the variable was 0.5. All the coefficients resulted positive as might be expected.

Table 6.26. Most significant variables affecting the distance to reemerged roadway on DSM01.

Rank	$h_2 = 0.2$ m		$h_2 = 1.1$ m	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive	$L_{in}+L_0+L_m$	Positive
2	D_{crests}	Positive	D_{crests}	Positive
3	$K_{V,1}$	Positive	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive
4	$K_{V,0}$	Positive	L_m+L_1	Positive
5	θ_0	Negative	$K_{V,1}$	Positive

6.2.4.2.5. Effect of alignment variables on the area of hidden sections in graph

In this subsection the results of the linear regression models that predicted the effects of alignment variables on the area of hidden sections in the sight distance graph are presented. These results are exhibited in Table 6.27, where identical patterns can be noticed between the sets of results of both target heights. Different parameters appear such as difference in grades sum of element lengths and roadway width. The lengths that best adjusted are those of the square root values, indicating the usual less-than-proportionally increase rate. The R^2 value remained as low as 0.22 in the variables that had the best fit. The coefficients were positive as expected.

Table 6.28 shows the basic variables that had the lowest p-values and performed the individual best fit. However, those goodness-of-fit indicators reached 0.11 at most in the lowest target

height and as little as 0.04 in the case of the highest target height. In the first ones, the terms are varied, yet they did not include horizontal deflection angles. The signs of the estimates, which are positive in any case, are coherent with what might be expected, except for the case of the inverse of the sag curvature. Such an estimate shows an effect that is opposed to that showed for other parameters (e.g. distance to reemerged roadway). In the second ones the terms are referred to lengths and the estimates have positive coefficients.

Table 6.27. Most significant variables affecting the area of hidden section in graph on DTM01.

Rank	$h_2 = 0.2 \text{ m}$		$h_2 = 1.1 \text{ m}$	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	θ_{sags}	Positive	θ_{sags}	Positive
2	$L_0+L_m+L_1$	Positive	$L_0+L_m+L_1$	Positive
3	W_{RW}	Positive	W_{RW}	Positive
4	L_{in}	Positive	L_{in}	Positive
5	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive

Table 6.28. Most significant variables affecting the area of hidden section in graph on DSM01.

Rank	$h_2 = 0.2 \text{ m}$		$h_2 = 1.1 \text{ m}$	
	Parameter	Sign of coefficient	Parameter	Sign of coefficient
1	L_{in}	Positive	L_{in}	Positive
2	$K_{V,1}^{-1}$	Positive	$L_{in}+L_0+L_m$	Positive
3	θ_{sags}	Positive	$L_{in}+L_0$	Positive
4	$L_0+L_m+L_1$	Positive	D_{crests}	Positive
5	W_{RW}	Positive	$L_{in}+L_0+L_m+L_1+L_{out}$	Positive

6.3. Development of standard spatial elements for motorways

As described in the literature review, SSE are alignment segments where the ends of their horizontal and vertical projections approximately coincide. They were adopted by the current German design standards for rural roads as a technique to prevent perspective deficits in the spatial alignment of single carriageways. The major safety-relevant deficits on rural roads comprise hidden beginning of curves and large sight-hidden dips. In principle, the presence of sight-hidden dips, sections where drivers may be misled during passing maneuvers, do not involve such an evident hazard on motorways as that existing on single carriageways. A special issue on the spatial alignment of motorways are hidden sections caused by median barriers concurring with an unfavorable superposition of horizontal and vertical alignment.

6.3.1. Methodology

The perspective recreation of the motorway was performed in CARD/1, according to the German visualization guidelines (FGSV, 2008a). As abovementioned, median barriers produce a visibility reduction, giving rise to possible discontinuities in the view of the roadway on motorways. Thus, the cross section of the virtual motorway sketched includes these features.

The following spatial alignment deficits were sought and recognized:

- Hidden beginning of curves
- Sight-hidden dips
- Hidden dips with jumping
- Median barrier discontinuities
- Broken back effect
- Sharp bends (horizontal, vertical and both)
- Tail motion

The evaluation of the spatial alignment based on safety performance is cumbersome owing to the high variance in data, the multiple factors involved in accidents and because their link to safety is not as clear as for other factors related to the design.

When seeking for guidelines to the arrangement of SSE based on offsets between stitching points, the possible influence of alignment recreation of in-service highways is subject to discussion. Little deviations on alignment parameters may cause large displacements of the stitching points between them. If the criteria for the definition of SSE are based on the relative position of the element ends, the alignment recreation procedure might be robust.

The assessment of the virtual perspective view followed also quantitative criteria, using the sight distance graphs. Measurable parameters of alignment deficits provide the basis of the quantitative criteria to develop the MSSE.

First, the alignment was split into segments so that no more than one element on each projection is comprised. Then, the occurrence of deficits was sought and detected using the highway perspective simulator of CARD/1.

To find patterns at the stitching points between alignments that discern good and poor spatial coordination, several variables were considered:

- Offset (m) horizontal/ vertical element
- Offset (% length) horizontal/ vertical element
- Overlapping length (m)
- Overlapping length (% length) horizontal/ vertical element
- Non-overlapped length (m)
- Central point offset (m)
- Central point offset (% length) horizontal/ vertical element

In addition, the effect of other alignment parameters was studied:

- Horizontal curve radius
- Horizontal curve deflection angle.
- Vertical curve parameter
- Element length

The range of a spatial alignment deficit can be quantified in the sight distance graph. It was assumed to affect the sub-section from the alignment segment where the deficit is perceived to

the element where flawless visibility is recovered, if applicable, or the further element visible otherwise.

Leutner (1974) stated that the driver immediate action on rural roads is determined by the stretch ahead that the driver is going to travel over the next 3 s at a speed of 90 km/h. This criterion has been extensively considered in the German normative framework. Federal motorways adopt a design speed of 130 km/h or higher, thus the corresponding distance of 110 m may be taken for the immediate action stretch.

6.3.2. Results and discussion

First, the criteria of H ViSt about SSE detailed in section 2.4.3.3 was extrapolated to motorways. It was found that few SSE exist in the alignment of the motorway studied. Whereas only 3 isolated vertical SSE were detected, 24% of the horizontal element ends are arranged so that the offset is smaller than 20% of their length. In the case of profile elements, 5 isolated SSE were found whilst the share of offsets below threshold decreased down to 14%. As a result no sequence of SSE was found.

To research the possible influence of variables listed in section 6.3.1, logit regression models were adjusted. Each variable was analyzed alone to avoid multicollinearity. None of the variables resulted a significant contributor to the avoidance of spatial alignment deficits. In the counterpart analysis of the alignment parameters, it was nonetheless observed that larger and smoother alignments yield indeed much less deficits in the perspective. The accomplishment of the SSE requirements of H ViSt is quite demanding. These criteria are stringent and do not guarantee alone the removal of perspective deficits. Also, locations where not a single SSE was found showed flawless spatial alignment coordination. Thus requirements for motorway standard spatial (MSSE) can be relaxed with respect to the SSE specified in H ViSt, or hardened where reported to be less effective. The following guidelines have been drafted according to general patterns found.

To use a percentage of the element length as criterion to determine the possible offset of between element ends has two main advantages. First, it is simple and second, it might be numerically valid for every element length. However, this criterion is awkward when it comes to large elements. In the sample analyzed, around 35% of horizontal alignments exceed 1 km long, which means 200 m offset on either end might be allowed. This fact yields a noticeable intrusion of segments that may actually create unfavorable superposition even though the requirements of SSE are fulfilled. As general threshold, the shorter length of 110 m or 20 % of the length of the horizontal alignment parameter is proposed as maximum offset, in accordance with the immediate action zone criterion. However, some important exceptions are proposed below.

With regard to short alignments, they should be definitely avoided because basic perception reasons, rather than superposition. This fact highlights that spatial alignment coordination criteria cannot be considered as surrogate design criteria for SSD, speed or consistency, but a compliment to ensure the absence of deficits.

Not only the definition of isolated elements is important, but also the appropriate choice of sequences. Alignment element sequences should not be left to chance. For example, the coincidence of vertical curves with horizontal curves enhances perception of large radii curves.

Concerning restrictions that can be relaxed, horizontal curves overlapped with a sequence grade – sag – grade revealed good appearance qualities, as long as the features of the elements involved comply with SSD dimensions. If the horizontal curve does not overlap the three profile elements, then it should be limited by the ends of the sag.

Offsets on crests and sags should not be interchangeably allowed on both directions because hidden beginning of curves, lengthening and shortening can be produced. For that reason, the offsets of Table 6.29 and Table 6.30 are proposed where crest curves and horizontal bends are overlapped, devised similarly to those in Table 2.2 and according to the developments of Zimmermann (2001). They are based on the fact that a driver must foresee a horizontal bend from a distance greater than the abovementioned immediate action zone (110 m in the case of motorways) before entering on it (Figure 6.7). The relevant change in direction of 3.5 gon is taken also from the German normative framework (FGSV, 2008a). Again the fact that alignment parameters on motorways are larger comes into play. An implementation criterion was taken from decision sight distance in the AASHTO guide (2011): Avoidance maneuver C (speed/path/direction change). As a distance of 390 m is required, no offset greater than 280 m should be needed to avoid hidden beginning of curves.

For the relevant change of direction, the length of the spiral L_C as a function of its parameter A is given by:

$$L_{Sp} = A \cdot \sqrt{\frac{3.5 \cdot \pi}{100}} \quad (6.1)$$

As the German standard for motorways (FGSV, 2008a) sets the observer's eye height at 1 m and the target height right on the pavement surface, the line of sight on the vertical projection yields:

$$h_1 = \frac{-L_2^2}{2 \cdot K_v} - \frac{L_2}{K_v} \cdot L_1 \Rightarrow 0 = L_2^2 + 2 \cdot L_2 \cdot L_1 + 2 \cdot K_v \quad (6.2)$$

Resolving the equation:

$$L_2 = -L_1 + \sqrt{L_1^2 - 2 \cdot K_v} \quad (6.3)$$

The offset can be therefore expressed as follows:

$$Offset = L_{Sp} - L_2 = A \cdot \sqrt{\frac{3.5 \cdot \pi}{100}} + L_1 - \sqrt{L_1^2 - 2 \cdot K_v} \quad (6.4)$$

$$\Rightarrow A \cdot \sqrt{\frac{3.5 \cdot \pi}{100}} + 110 = \sqrt{(110 + Offset)^2 - 2 \cdot K_v} \quad (6.5)$$

Resolving the equation:

$$Offset = -110 + \sqrt{12100 + 2 \cdot K_v + A^2 \cdot \frac{3.5 \cdot \pi}{100}} + 220 \cdot A \cdot \sqrt{\frac{3.5 \cdot \pi}{100}} \quad (6.6)$$

If the transition is produced without a spiral, then the offset reads:

$$Offset = -110 + \sqrt{12100 + 2 \cdot K_v + \frac{3.5^2 \cdot R^2 \cdot \pi^2}{200}} + 220 \cdot \frac{3.5 \cdot R \cdot \pi}{100} \quad (6.7)$$

Where the preceding alignment element is a sag instead of a grade, or a curve instead of a tangent, this criterion lays on the safe side.

Table 6.29. Minimum offset required for spiraled horizontal curves on crests on motorways.

Vertical curve parameter, K_v [m]	Spiral parameter A [m]						
	100	150	200	250	300	400	500
-2000	12.7	64.9	51.4	69.7	87.6	122.8	157.3
-3000	0.5	22.8	43.2	62.6	81.3	117.6	153.0
-4000		11.9	34.4	55.1	74.8	112.4	148.6
-5000			24.9	47.2	68.0	107.0	144.1
-6000			14.3	38.7	60.8	101.4	139.4
-7000			2.3	29.6	53.2	95.6	134.7
-8000				19.5	45.2	89.7	129.9
-9000				8.3	36.5	83.5	125.0
-10000					27.2	77.0	119.9
-12000					5.3	63.3	109.3
-15000						39.5	92.3
-20000							59.1

Table 6.30. Minimum offset required for non-spiraled horizontal curves on crests on motorways.

Vertical curve parameter, K_v [m]	Curve radius, R [m]						
	720	800	900	1000	1300	1500	2000
-2000	79.7	93.6	111.3	129.2	183.9	221.0	314.7
-3000	74.4	88.7	106.7	125.0	180.5	218.0	312.4
-4000	68.9	83.6	102.1	120.7	177.1	214.9	310.0
-5000	63.2	78.3	97.3	116.3	173.5	211.8	307.6
-6000	57.3	73.0	92.4	111.9	170.0	208.7	305.2
-7000	51.3	67.4	87.4	107.3	166.4	205.5	302.8
-8000	44.9	61.7	82.3	102.7	162.8	202.3	300.4
-9000	38.3	55.7	77.0	97.9	159.1	199.1	297.9
-10000	31.4	49.6	71.6	93.0	155.3	195.9	295.5
-12000	16.5	36.5	60.2	82.9	147.7	189.3	290.5
-15000		14.4	41.6	66.7	135.8	179.1	282.9
-20000		No offset	3.9	35.7	114.5	161.2	270.0

Figure 6.7. Alignment projections on hidden beginning of curve.

Unlike in the revision of SSE criteria from H ViSt to RAL, a sequence of two elements on a single grade should not be allowed in all cases. It results particularly unfavorable where a sequence tangent-curve are combined with a grade that is connected to a sag curve and an opposite-hand curve. This combination of elements rises visual discontinuities in all cases caused by median barriers (Figure 6.8). Conversely, if the grade is delimited by two crests, the superposition of as many horizontal elements as desired results satisfactory.

Very smooth crests can be also split into several MSSE as allowed in the case of grades. The vertical curvature threshold value should be no smaller than 25,000 m. Similarly, smooth enough sags should be allowed to split into several MSSE according to the superimposed horizontal alignments. However, the boundaries of this criterion are not clear so no recommendation is given in this regard.

Unless reemerged sections occur far enough from driver, sight-hidden dips disorientate or lead them to misjudge the actual course of the roadway. If the alignment is arranged as a sequence of SSE, it is virtually impossible that a straight sight-hidden dip is formed. Nonetheless, sight-hidden dips with jumping are still likely to occur (Figure 6.9). For these reasons, it is discouraged that a sequence crest – sag without intermediate grade is overlapped with a tangent-curve sequence, even though offsets between element ends comply with specifications.

Figure 6.8. Visual discontinuity of the roadway caused by median barrier.

Figure 6.9. Sight-hidden dip with jumping.

6.4. Concluding remarks

The consistent appearance of a highway affects, directly or indirectly, the driver's behavior and their driving performance. As geometric design standards are frequently limited to descriptive guidelines with regard to design deficits, especially sight-hidden dips, this part of the thesis aimed at providing more defined design criteria to avoid the existence of such deficits or to tackle the redesign of the alignment.

First, the effects that elevation model types have on the characterization of sight-hidden dips on two-lane rural highways have been disclosed. The sight distance evaluation was made with two target height values 0.2 m and 1.1 m. The first one evaluates if hidden interim sections are produced in the perspective whilst the second one aims at detecting potential passing issues. The sample of hidden dips was segmented into diving and jumping sections, the effects being more noticeable on the latter ones. Next, the parameters that most likely affect the existence of hidden dips and their characteristic parameters have been identified. Not in all cases the results delivered were highly revealing. In particular the models adjusted to predict the existence of hidden dips could not satisfactorily explain the matter probably due to other variables (hard to be identified and quantified) not contemplated in the models that greatly

affect the phenomenon. The models might therefore be affected by underlying variance. This fact highlights the difficulties in addressing such shortcomings in design.

Moreover, the variables that had higher goodness-of-fit values oftentimes were the lengths of vertical alignments and the roadway width, the effect of both being decreasingly proportional. These values can hardly be altered given the topography (for example in dells) and the basic design factors. On the basis of this, the horizontal alignments should be redesigned instead or, if there are major right-of-way restrictions, the algebraic differences in grades should be reduced, preferably those by the sag.

Very few SSE were detected on the crest – sag sequences studied. The presence of SSE was not found to be relevant to prevent the existence of hidden dips. However, the incorporation of such a variable in models may not lead to unbiased outcome given the abovementioned low incidence.

Practical rules to prevent the existence of shortcomings in the perspective that drivers have of the motorway are provided. To facilitate drivers to foresee horizontal curves overlapped with crests, the required offset values to prevent the existence of hidden beginning of curves are provided on the basis of decision sight distance. Also, the principles of SSE, in force in the German standards for rural highways, were reviewed for their implementation on motorways.

Chapter 7

CONCLUSIONS AND FUTURE RESEARCH

7.1. Conclusions

This thesis provides a methodology to analyze sight distance on highways and 3D alignment coordination, studying their relation to traffic safety. In this chapter, a summary of the conclusions of the research are presented following the structure of the research objectives formulated in section 1.4.

7.1.1. Study of 3D sight distance

7.1.1.1. Creating a cartographic database

The potential and capabilities of GIS have been exploited to produce the necessary inputs for the estimation of sight distance on highways under a 3D approach, the computation of sight distance itself and the analysis of results. Besides GIS-based *Road Sight Distance* software, several spatial analysis tools were utilized, including those necessary to derive the vehicle trajectories, to process the elevation models or to match accidents to the curves where they took place.

This proves the potential and versatility of GIS for the integrated analysis of highways. The analysis can cover several aspects on a single platform, such as traffic volume, speed (design, posted, or operating), design consistency, ASD and 3D alignment coordination. The location of accident spots, regardless of whether they are provided through their coordinates or through linear reference on the highway (km and hm), can be carried out at ease. Moreover, the analysis of complex problems is improved, as in those related to the geometric design of roads and traffic accidents. Looking ahead, the fact that all the information is on a single platform simplifies data management and updating.

This research has shown that several variables useful for safety performance studies could be determined: speed profile, difference between speeds of adjacent elements, ASD, lengths of road sub-sections that can (and cannot) be seen from each position, and the distance from which a particular position of the road is seen. Road network administrators may benefit from this methodology to address the problem of traffic safety caused by possibly poor geometric design.

The guidelines proposed to create a cartographic database save some work in the data processing that had to be done manually before and now this work is semi automated.

7.1.1.2. Effect of the elevation models on sight distance

3D sight distance has been studied using three different high resolution LiDAR DEM: DTM's and DSM's from airborne survey and the MMS DSM from terrestrial survey. DSMs obtained from surveying flights are more affordable since these may have been produced previously for any general purpose, while terrestrial surveying requires deploying a specially equipped vehicle to survey the road. However, a terrestrial MMS provides a greater density of points surveyed; hence the resulting surface model depicts the roadway and its roadsides more accurately. Additionally, the points on the grid obtained through the MMS are aligned with the road centerline, a feature that facilitates a more homogeneous representation of the ground surface and the elements on it. The resolution of DSM01 is constant (1 m spaced grid in this study), resulting in a more regular grid. In contrast, the resolution obtained through terrestrial MMS is variable, diminishing and becoming more irregular as the surveyed points are further from the scanner. Nonetheless, in sight distance studies, both the resolution and the area covered by MMSs have proven to be sufficient.

The effect of elements by the roadsides on sight distance results, depending on the DEM employed, has been observed, highlighting the strengths and weaknesses of the use of each elevation model. When the road analyzed is straight, it is simpler to use a DTM, not only because the results are identical (if the overhanging elements are removed according to the procedure hereby described), but also because the cloud of points does not have to be processed to remove points corresponding to non-static items or overhanging elements. Such an analysis using a DTM is sufficiently reliable if no trees or other overhanging structures affecting sight distance are present above the roadway. However, in sections with curves, where the lines of sight are projected out of the pavement surface, differences between the models studied are evident. In such cases, the outcome is more tied in with reality if any of the DSMs is utilized. It is essential to take into account obstacles by the roadsides (such as trees or buildings) in sight distance analysis. Thus, the use of DTMs is not recommended where the alignment is sinuous in the horizontal projection and dense vegetation or other perceivable elements arise by the roadsides. In general, DSMs are more suitable for ASD estimation and other studies such as 3D alignment coordination because results obtained using a DSM are more realistic.

For DSM derived from airborne LiDAR in particular, data require much less processing and the interpretation of the output is easier as these models do not usually comprise roadside equipment. However, analyzing sight distance on these models is sometimes entangled by the impossibility of representing the surface of the scene due to Delaunay triangulation limitations.

The MMS DSM allows maximum accuracy when compared to other models derived from aerial LiDAR, since it reproduces the road and its environment in a much more realistic way. However, the high sensitivity of this technology leads to capture points of elements by the roadsides, even small ones, creating significant problems in both pre-processing the cloud of points as well as the study of issues related to alignment coordination. The use of these models requires a much more thorough analysis to discern whether the cause of insufficient sight distance is road geometry and landform (i.e. cut-side slopes), or conversely, it is due to an item by the roadsides.

The final ASD value is indeed very sensitive to little sight distance obstructions, such as traffic signs or isolated trees. This fact is observable when sight distance was studied using the MMS DSM.

The use of models derived from MMS DSM is recommended to study actual sight distance conditions more accurately for very specific cases, in absence of circumstances that may cause some of the problems inherent in the use of Delaunay triangulations explained in this document (the algorithm employed by GIS to create a TIN does not allow overhanging elements on the plane). However, this issue is successfully solved by the processing method explained in section 3.1.3.3. In addition, the ASD reduction caused by road equipment such as metallic barriers has been evidenced in these models, although in this particular case, the result is sensitive to the initial parameters (driver's eye height and target height). Also, it is recommended to identify elements by the roadside that could lead to misleading results, such as fences, to interpret the results appropriately. Finally, although the DSM can be considered more reliable to recreate the real sight distance, the fact that vegetation vary depending on the season may be a limitation to the use of DSM.

A paired comparison was performed between the 3D ASD results derived from the three elevation models (DTM01, DSM01 and MMS DSM) on two of the selected highways. The Wilcoxon test showed that the paired ASD values that resulted from the ASD estimation were significantly different. A preliminary assessment of the SSD on those highways was performed, and the uncertain results highlighted the need for a thorough study of SSD that considers this mode of failure with greater detail and under a probabilistic approach.

7.1.2. Effect of sight distance on highway safety

7.1.2.1. Reliability analysis of sight distance

The main objective of this part was to investigate the safety implications of modelling ASD by means of reliability analysis. It advocates the consideration of uncertainty in highway geometric design rather than the usual deterministic approaches. Indeed, probabilistic approaches adapt well to research where random variables are involved in very diverse fields.

In the first part of the reliability analysis, the ASD and the SSD were studied using a Monte Carlo simulation on one of the selected highways. A numeric algorithm based on the finite element method has been developed to compute the distance travelled during a stopping maneuver where the grade is variable. First, the average effect of driver's eye height and target height on the ASD output was quantified in the case study. It was found that areas in which the P_{nc} resulted higher are located at or near the alignment elements that have tighter parameters: curves with smaller radius, crests with smaller radius and steeper downward grades. The methodology proposed has thus been validated successfully. The results yielded in this study also support the consideration of passenger car features for sight distance geometric design. When studying ASD on highways, the usual assumptions can be safely made by considering passenger cars only as the P_{nc} patterns were similar between passenger cars and heavy vehicles, and the higher P_{nc} values arose for passenger cars.

In the second part, the ASD of 402 horizontal curves, located in twelve two-lane rural highways was analyzed. Three ASD estimation methods were used: 1) 2D ASD, 2) 3D DTM ASD, and 3) 3D DSM ASD. The ASD results obtained through 2D and 3D methodologies were compared.

Overall, the ASD comparison results showed that there is a significant difference among the three ASD modelling methods in most cases. This emphasizes the importance of using the 3D modelled sight distance, particularly the 3D DSM ASD, either in highway design or during the service life as it is considered more realistic than the 2D one.

The studied curves were classified into different groups based on the existing roadside features or geometric elements that hinder driver's visibility. The resulted change in the estimated ASD between the three modelling methods under different conditions can be identified as follows:

- For curves with vegetation by the roadside, a significant difference is identified when comparing the three methods. The 3D DTM overestimates the ASD comparing to the other two methods. The concurrence of both alignment projections (i.e. horizontal and vertical projections) and the vegetation make the 2D approach overestimates the ASD when compared with the 3D DSM.
- For horizontal curves with cut side, only the pair that compares the two 3D methods has a significant difference.
- For horizontal curves overlapped with a crest vertical curve, the difference is significant when comparing the 2D approach with the 3D ones. The 2D method overestimates the ASD due to disregarding the 3D features such as the effect of superelevation, the existence of other near vertical curves, or the overlapping with a crest vertical curve.
- For curves located in clear highway segments, the results showed the greatest differences. The 2D model greatly underestimates the ASD when compared to the other two 3D models. The 3D DTM overestimates the ASD in comparison with the 3D DSM model.

Reliability theory was utilized to evaluate the risk level (P_{nc}) associated with limited sight distance for each ASD modelling method. In order to investigate the significance of the safety implications of the ASD modelling approach, the P_{nc} results were compared. The models that overestimated the ASD underestimated the P_{nc} . Overall, the result showed a significant difference in the estimated P_{nc} values between the three ASD modelling methods. This emphasizes the significant implications of using different ASD modelling methods in evaluating the risk associated with a specific design.

The significance of the differences in the P_{nc} values is generally higher than the significance of the differences in the estimated ASD values. For example, some compared pairs are associated with a non-significant difference in the estimated ASD, and, at the same time, have a significant difference in the estimated P_{nc} values at 95% confidence level. This can be explained by the non-linear effect of the estimated ASD value on the value of P_{nc} estimated through the reliability analysis.

The results emphasize the importance of considering the 3D modelled sight distance, particularly the 3D DSM ASD, when evaluating the associated risk either in highway design or

during the service life. In addition, the safety implications of the ASD modelling approach can be significant.

The interaction between the ASD and the operating speed must be borne in mind. The effect of the ASD modelling method is not only owing to the ASD itself but also to the speed in the profile at the location where it is produced.

Reliability analysis applied to geometric design represents a proactive measure to enhance facilities. Learn from safety performance to devise guidelines and standards for brand new highways or to upgrade the existing ones. It provides, moreover, a surrogate tool for CMF on design where such factors are not available.

Finally, the effect of grade on the P_{nc} was analyzed. The impact of grade is more significant as the P_{nc} is closer to 0.5. It was found that the negative effect of a downhill grade is greater than the counterpart positive effect of the opposite uphill grade on the P_{nc} . Even for low probability values, the effect of grade is important inasmuch as the grade rises. Therefore, it is highly encouraged that reliability-based SSD analysis considers the grade.

7.1.2.2. Accident frequency modelling

Safety research aims at establishing quantitative relationships between various planning and design variables using SPFs and collision modification factors CMFs. However, in view of the difficulty for using CMFs in many complex design situations, surrogate tools must be devised. In this research, an evaluation of safety performance based on the reliability results was carried out. A quantitative relationship between collisions and operating and design variables was established. The risk exposure was contemplated by the incorporation of the AADT and the curve length while the P_{nc} quantified the safety level of design. The variables included in the model resulted significant at the 95 % confidence interval, the exposure variables involving a greater effect on accident frequency. The models developed should help establish linkage between collision frequency and design variables which would lead to a wider application of reliability-based geometric design and to admit reliability based design into traditional benefit-cost analysis.

7.1.3. Evaluation of the 3D alignment coordination

7.1.3.1. Study of sight-hidden dips on two-lane rural highways

A sight-hidden dip may be a safety issue for passing maneuvers where the ASD is less than the required PSD. The effect of sight distance modelling on the characteristic parameters of sight-hidden dips was studied (range, longest hidden section and distance to reemerged roadway). A new parameter was furthermore defined, namely the area of hidden sections in the sight distance graph. A computation algorithm based on line-of-sight loops performed on two different elevation models, which already demonstrated good performance in the analysis of ASD on in-service highways, has also helped in this study. Two target heights were assumed, 0.2 and 1.1 m. The lower one aimed at detecting hidden interim sections in the sight distance analysis while the higher one focused on hidden sections that may compromise safety in passing maneuvers.

Overall, it was found that the differences between parameter values measured on either elevation model were statistically significant. If the sample is segmented into two groups, namely diving sections (horizontal projection prevalently straight) and jumping sections (curved horizontal projection), the results of the statistical analysis vary slightly. Whereas the differences between parameter values remain significant in jumping sections, the null hypothesis could not be rejected in diving sections for the longest length of interim hidden section and the distance to the reemerged roadway under the assumption of target height 1.1 m. This means that the use of DSMs is highly recommended, particularly in curved sections, to evaluate sight distance also in the assessment of sight-hidden dips and their potential effects on safety.

The effect of alignment variables on the occurrence of sight-hidden dips and on their fundamental parameters was also studied. The relationships found with the probability of these alignment shortcomings occurring seemed to be affected by high variance. Modelling these effects using a DTM would match cases where roadsides are clear and the use of a DSM would correspond to sections with sight obstructions by the roadsides. In general, the variables devised to measure alignment coordination (particularly deflection angles turned on vertical elements) explain to a certain extent the existence of sight-hidden dips on DSMs.

The models of sight-hidden dip parameters resulted more effective than those aiming at predicting sight-hidden dip occurrence. The models adjusted to assess the effects of alignment on hidden sections deeper than 1.1 m (potential passing issues) had lower goodness of fit, they thus explaining the dependent variable less satisfactorily. Certain models where no variable had high goodness of fit are likely to be affected by high variance. This fact highlights the difficulties in addressing such shortcomings in design.

In general, the lengths of vertical alignments were the variables that most frequently provided satisfactory explanation for the dependent variables, especially for lengths of hidden sections, distance to reemerged roadway and area of hidden sections in the sight distance graph. The roadway width was also found to be a significant factor favoring both the existence of hidden dips and several of the characteristic parameters. However such a design parameter cannot be changed as it is determined by project specifications and has unquestionable positive effects for highway operation and safety. From the effect that roadway width has on hidden dips, it can nonetheless be inferred that the existence of jumping sections is largely affected by the available clearances.

The incorporation of SSE in the models did not yield satisfactory results probably owing to the low frequency of these elements on the crest – sag sequences studied. Actually, no sequence of SSE was found in the observations.

Finally, it is worth noting that most of the selected two-lane highways analyzed in this thesis had underwent alignment upgrades. Generally, the alterations in geometrics comprise the horizontal projection and the cross section. Except for bypass sub-sections, very rarely is the vertical alignment also redesigned. This common practices yield 3D flaws in the perspective, such as those described in the literature review. Moreover, this problem was observed in the sample at many sites when the 2D ASD was estimated (Chapter 5). Where horizontal curves and crests were overlapped, the vertical 2D ASD resulted shorter than the horizontal counterparts, violating in some cases the guidelines for alignment coordination of the Spanish standard. This

thesis, therefore, advocates the consideration of redesigning also the vertical alignment to meet the corresponding standard on highways in alignment improvements on the basis of cost – benefit analysis.

7.1.3.2. Study of spatial alignment coordination of motorways

The study on motorways delivered guidelines for the enhancement of spatial alignment coordination through the use of MSSE. Strengths and weaknesses of the current SSE definition in H ViSt were noticed. In general, the principles on SSE of the German design standards for rural roads can be maintained on motorways. However, several exceptions and corrections were made since motorways are characterized by larger alignment elements and wider cross-sections. On the one hand, the overlapping length must be limited up to 110 m. On the other hand, smooth elements should be able to be split into several MSSE, and certain sequences can be introduced as they were found not to involve visual shortcomings.

Finally, it is recalled that spatial alignment coordination criteria should not replace more basic design criteria based on speed and sight distance, namely dimensions and parameters of horizontal and vertical elements. Where hidden dips are produced, the alignment arrangement according to SSE would not resolve an insufficient SSD issues on isolated segments unless these alignments comply with SSD standards (for example a crest on a tangent).

7.1.3.3. Recommendations for the 3D alignment coordination of highways

7.1.3.3.1. Sight-hidden dips on two-lane highways

Based on the results of this research, general recommendations for the redesign of sections where sight-hidden dips are produced are delivered. If the removal of the perspective shortcoming is pursued, the horizontal alignment is the most relevant factor to focus on. Horizontal deflection angles were found to be important features to avoid sight-hidden dips on the DSMs, which are thought to display the real shape of the road and its roadsides in the most realistic way. Therefore, if there are no major restrictions on the horizontal projection, a horizontal curve may be inserted in the alignment to prevent the occurrence of such a shortcoming. Particularly, the most effective one would overlap the interim grade as Ω_m and Ω_1 were the most frequently found significant basic variable in the logit models on the DSM01. If the depth of diving is to be reduced only, the reduction of algebraic differences in grades at crests and sags is the most effective measure.

The results of the analysis in real cases showed opposite effects of increasing the sag vertical curve parameters. While larger parameter values produce a significant increase of the length of hidden sections and of the distance to reemerged roadway, they reduce the area of hidden sections on the sight distance graph.

If a reduction of the hidden dip range is pursued, the reduction of the algebraic difference in grades of the sags are necessary on clear environments and diving sections, whereas it would be more effective to reduce the difference in grades at the crest on areas with low clearances. The reduction of the range is preferable rather than to seek for the reduction of the other characteristic parameters. The other characteristic parameters are largely influenced by the vertical alignment lengths, which in very few cases might be altered if they are determined by a

dell in the topography. Overall, the reduction of the algebraic differences in grades is more effective than the changes in the vertical curvature parameters as the first ones also reduce the parameters of the hidden areas that may entail risk at passing maneuvers (target height over 1.1 m).

7.1.3.3.2. Spatial alignment of motorways

With regard to MSSE, the non-overlapping distance (20% of the horizontal alignment element length) between elements should not exceed 110 m. Next, separated MSSE can be considered on a single crest if its curvature at the vertex exceeds 25,000 m. The same consideration is valid on grades delimited by two crests. A sequence crest – sag without intermediate grade overlapped with a tangent-curve sequence, even though offsets between element ends comply with specifications are discouraged.

In addition, a quantitative-based criterion to avoid hazardous hidden beginning of curves on motorways was proposed when bends on both projections are overlapped (Table 6.29 and Table 6.30).

7.2. Contributions

This thesis has contributed to increasing knowledge on highway design and safety through the following facts:

- A methodology to perform sight distance studies on highways has been developed and validated. It is valid for the analysis of sight distance both on brand new highways and on in-service highways.
- Guidelines for creating an appropriate cartographic database, including input data processing, in order to perform the sight distance calculation have been delivered.
- The consideration of DSM's in sight distance studies has been backed since these elevation models provide, in general, a more lifelike depiction of the roadway and especially the roadsides, comprising the relevant landscape features that determine sight distance.
- The study using Monte Carlo lays the foundation for a continuous-section probabilistic analysis of traffic safety and ASD on already-built highways.
- The methodology proposed may establish the basis for the continuous assessment safety analysis of highways in order to find possible relationships between ASD and accident occurrence.
- Although it had already been proved that 2D ASD estimation techniques may underestimate or overestimate the actual ASD, the effects of these differences on safety were still uncertain. By using the reliability analysis, the risk difference has been quantified, disclosing the non-linear effect that affects safety.
- A further step has been added into the direction of developing fully probabilistic (risk-based) geometric design standards.
- Reliability analysis has been applied to SSD using 3D ASD values and contemplating for the first time the actual grade of the stretch travelled during the hypothetical stopping maneuver.

- The relationship between reliability-based risk measures and predicted collisions has been established, where AADT, segment length and P_{nc} showed a statistically significant effect on safety.
- The present methodology provides an effective surrogate tool to evaluate cost effectiveness of sight distance improvements on highways as opposed to traditional highway safety analysis, which is highly exposed to variance.
- The effect of elevation models on the characterization of sight-hidden dips has been proved to be significant.
- Guidelines for the redesign of crest – sag sequences have been proposed in order to prevent the existence of sight-hidden dips or to reduce their potential effects on safety by reducing the dimensions of the characteristic parameters.
- A criterion to prevent hidden beginning of curves on motorways based on the alignment element parameters and their superposition has been proposed.
- MSSE have been defined in order to prevent deficits in the spatial alignment of motorways, mainly hidden intermediate sections caused by unfavorable superposition of alignments.

7.3. Future research

The research work carried out in the context of this doctoral thesis permitted to arrive at the conclusions and contributions described in the two foregoing sections. However, the findings presented open new avenues for research, the most noteworthy ones being listed below:

- Development of a fully-automated recreation of horizontal and vertical alignment procedure, capable of recreating complex successions of alignments like the ones found in the selected highways. Machine learning techniques and heuristic algorithms are proposed for this assignment.
- Development of a 3D sight distance estimation procedure able to effectively address overhanging structures such as cantilever structures, overpasses or tunnels.
- Incorporation of the 3D ASD results of the hereby proposed methodology as a predictive variable to develop a new operating speed model.
- Research the safety effects of spatial alignment coordination on large network data.
- Study of sight distance under nighttime conditions and analyze the outcome from the standpoint of traffic safety.
- Research the target level of safety in design through reliability analysis based on safety performance.
- Development of CMF for sight distance restrictions in curves based on reliability analysis and safety performance.
- Extend the reliability analysis to contemplate multiple modes of failure: SSD, vehicle stability (rollover) and vehicle skidding.
- Reliability-based analysis of nighttime SSD and comparison to the respective results of the daytime SSD analysis.
- Perform a comprehensive reliability analysis in Rt software incorporating multiple vehicle types (e.g. light and heavy vehicles as well as motorbikes) contemplating conditioned probability among the corresponding input variables.

- Network-based spatial analysis of safety performance incorporating reliability measures exploiting the capabilities of GIS's.
- Modelling sight-hidden dips using machine learning techniques by incorporating alignment element features.

Chapter 8

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Appendix I

SELECTED HIGHWAYS

M-104

Location

Feature	Initial end	Final end
Town	San Agustín de Guadalix	Colmenar Viejo
Access type	Roundabout	Roundabout
Kilometer	1+800	12+800

Characteristic alignment values

M-221 / M-222

Location

Feature	Initial end	Final end
Town	Carabaña	Estremera
Access	3-way intersection	3-way intersection
Kilometer	18+400	26+200
	14+000	19+300

Characteristic alignment values

M-325

Location

Feature	Initial end	Final end
Town	Colmenar de Oreja	Villamanrique de Tajo
Access	Urban street	Urban street
Kilometer	0+000	15+000

Characteristic alignment values

M-513

Location

Feature	Initial end	Final end
Town	Boadilla del Monte	Brunete
Access	Roundabout	Roundabout
Kilometer	7+800	16+700

Characteristic alignment values

M-600

Location

Feature	Initial end	Final end
Town	San Lorenzo del Escorial	Villanueva de la Cañada
Access	Roundabout	Roundabout
Kilometer	10+700	27+600

Characteristic alignment values

M-601

Location

Feature	Initial end	Final end
Town	Collado Villalba	Navacerrada
Access	Roundabout	Junction
Kilometer	1+800	12+200

Characteristic alignment values

M-607

Location

Feature	Initial end	Final end
Town	Colmenar Viejo	Cerceda
Access	Motorway junction	Roundabout
Kilometer	35+900	48+200

Characteristic alignment values

M-610

Location

Feature	Initial end	Final end
Town	Bustarviejo	Miraflores de la Sierra
Access	Urban street	Urban street
Kilometer	11+500	19+800

Characteristic alignment values

M-611

Location

Feature	Initial end	Final end
Town	Soto del Real	Miraflores de la Sierra
Access	Roundabout	Urban street
Kilometer	1+900	6+900

Characteristic alignment values

M-629

Location

Feature	Initial end	Final end
Town	Miraflores de la Sierra	Canencia
Access	Urban street	Urban street
Kilometer	1+400	16+600

Characteristic alignment values

M-633

Location

Feature	Initial end	Final end
Town	Cabanillas de la Sierra	Valdemanco
Access	Roundabout	3-way intersection
Kilometer	0+000	6+200

Characteristic alignment values