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Relationship of predicted speed reduction on horizontal curves and safety in two-lane rural roads in Spain

Juan de Oña[1], Laura Garach[2], Francisco Calvo[3], Teresa García-Muñoz[4]

ABSTRACT

According to different studies, speed reduction is considered as one of the major factors in contributing road safety. For that reason, several guidelines have been recommended for maximum desirable speed reductions from tangents to horizontal curves and for maximum differentials between design and operating speeds on horizontal curves. The Interactive Highway Safety Design Model (IHSDM) Design Consistency Module presents an analysis of the relationship between speed reduction and crashes for horizontal curves on United States (U.S.) two-lane rural highways. This paper presents the relationship between speed reduction and crashes for horizontal curves on Spanish two-lane rural highways. A model for using regression analysis to predict crashes is presented. Exposure, curve length and difference in 85th percentile speeds (ΔV_{85}) between successive tangents and horizontal curves, and between successive curves are used. The model's coefficients were different from the ones obtained for U.S. highways, although the values of the goodness-of-fit criteria were similar. In addition, the relationship between crashes and difference in speeds is also analyzed, taking the difference in speeds as a speed differential not exceeded by 85% of the drivers traveling under free-flow conditions ($\Delta_{85}V$), instead of considering it as ΔV_{85}. The two models (ΔV_{85} vs. $\Delta_{85}V$) give very similar results.

Keywords: Road Safety; Speed Reduction; Horizontal Curves; Consistency; Two-lane Rural Highways; Poisson Distribution; Spain; V_{85}

INTRODUCTION

Road crashes are one of the most important problems in our today´s society because they affect many people. According to the World Health Organization (WHO), approximately 1.24 million people die every year on the world's roads, and another 20 to 50 million sustain non-fatal injuries as a result of road traffic (WHO 2013). More than 70% of all curve-related fatal crashes occur in two-lane highways (Harwood et al. 2000; McGee and Hanscom 2006). The mean crash rate for horizontal curves is about three times the mean crash rate for highways (Torbic et al. 2004). Considerable research has been undertaken to improve safety at horizontal curves, based on the assumption that taking them at lower speeds will cause fewer crashes (Lima-Allen County Regional Planning Commission 2011; Montella 2009; Robinson and Knapp 2009; Hallmark et al. 2012; Elvik 2013a). Lima-Allen County Regional Planning Commission (2009) reviews low-cost treatments to be applied at horizontal curves in order to

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address the safety problem they pose. Depending on a curve's features, they recommend specific treatments, such enhanced traffic control devices or rumble strips. Montella (2009) demonstrated that different treatment typologies, caused crash reductions. The most effective treatment was the installation of curve warning signs, chevron signs, and sequential flashing beacons along the curve. Robinson and Knapp (2009) evaluated the speed and crash impacts of three permanently installed Dynamic Curve Warning Signs (DCWSs) in three Minnesota counties. DCWSs are low-cost technology that may help drivers select an appropriate speed when approaching a horizontal curve. Hallmark et al. (2012) studied the effect of installing DCWSs at 22 curves on rural two-lane highways. The results of the speeds obtained over a 12-month period after the signs had been installed provided that they were very effective in speed reduction. Elvik (2013a) analyzes the effect of a temporary speed limit of 60 km/h that was associated with a reduction in the mean speed of traffic of about 7.5%.

To ensure road safety, we need to analyze road layouts from the project phase. One technique used to improve safety on roadways from the point of view of the infrastructure is to examine the consistency of the design. Design consistency refers to highway geometry's conformance to driver expectancy (Ng and Sayed 2004, Castro et al. 2008). Generally, drivers make fewer errors in the vicinity of geometric features that conform to their expectations than at features that violate their expectations. The worse the consistency, the more likely it is that drivers will be startled and a crash will occur. This paper analyzes the relation between consistency (based on speed reduction) and crashes, based on two-lane roads in Spain.

General approaches to the relationship between consistency and road safety

Past literature shows that safety on horizontal curves has been studied from a number of perspectives. In the 1970s, studies on the effect of road safety consistency on isolated curves began to be undertaken in the U.S. (Taylor et al. 1972; Stimpson et al. 1977). The studies analyzed the relationship between several operating parameters such as pavement width, pavement markings or shoulder and crashes for a limited number of horizontal curves. Subsequently, several studies (Datta et al. 1983; Terhurne and Parker 1986; Zegeer et al. 1990) identified the degree of curvature as one of the key variables for estimating the number of crashes. Datta et al. (1983) pointed out that the degree of curvature is the only variable that has statistical significance. The work by Terhurne and Parker (1986) identified Annual Mean Daily Traffic (AADT) and the degree of curvature as the best variables for estimating the number of crashes. The study by Zegeer et al. (1990) identified the following variables to have a significant effect on crashes: degree of curvature, roadway width, curve length, AADT, presence of a spiral, super-elevation, and roadside condition. Fitzpatrick et al. (2000) analyzed the relationship between several different roadway alignment indices and road safety. Of the candidate measures, three showed statistically significant relationships to crash frequency: ratio of an individual curve radius to the mean radius for the roadway section as a whole; mean rate of vertical curvature; and mean radius of curvature.

Khan et al. (2012) have explored the relationship between safety at horizontal curves and sign types, specifically curve and turn signs. The crash prediction models on different types of roads displayed highly significant variables that showed a positive relationship with AADT, posted speed, and curve length, and a negative relationship with curve radius. Elvik (2013b) compared the relationship between crashes and various road features, such as radius and length, in 10 countries. He found that when the distance between curves was shorter, crashes diminish. Concerning curve length, the results varied from one country to the next. Some studies indicated that shorter curves have higher accidents that longer curves, some studies indicated the opposite and some studies indicated that accident rate in curves is unrelated to the length of the curve.

Quddus (2013) explored the relationship between mean speeds, speed variations and crash rate. The results suggest that mean speeds are not associated with crash rate when controlling for other factors affecting crashes such as traffic volume, road geometry and number of lanes. However, speed variation is found to be statistically and positively associated with crash rate.

Another approach has been to analyze whole sections of roads (Polus 1980; Mattar-Habib et al. 2008; Camacho-Torregrosa et al. in press). Polus (1980) analyzed the relationship between several different roadway alignment indices and crashes. His main conclusion was that safety on the roads increased as the indices became more uniform along the sections. In the same vein, for roads in Israel and Germany, Mattar-Habib et al. (2008) related the number of crashes to road consistency, using the integration consistency index by Polus et al. (2005), which considers the global variation of speed along a section of road and results in a single consistency value for the entire section. Using the same methodology, Camacho-Torregrosa et al. (in press) recently proposed a model that relates the integration consistency index to the crash rate for Spanish roads.

Consistency analysis based on speed reduction

The operating speed (V_{85}) is defined as the 85th percentile of the distribution of speeds by drivers in free-flow conditions on a particular location of the road alignment (Bella 2007). The consistency of an isolated geometric element of an aligment can be measured by comparing the element's operating speed with the design speed or by comparing the operating speed of successive elements.

Several guidelines (Babkov1968; Leisch and Leisch 1977; Kanellaidis et al. 1990) have been recommended for maximum speed reductions from tangents to horizontal curves and for maximum differentials between design and operating speeds on horizontal curves. In Russia, Babkov (1968) concluded that consistent and safe designs could be obtained when the difference in operating speed between two consecutive elements did not exceed 15% of the speed in the preceding element. In the U.S., Leisch and Leisch (1977) recommended that the difference in operating speed between two consecutive alignments should not exceed 15 km/h. Kanellaidis et al. (1990) suggested that good designs could be attained when the difference in operating speed between a straight section and the next curve does not exceed 10 km/h.

Park et al. (2010) introduced a new and easy to interpret speed consistency measure that simply represents the probability that a vehicle exceeds a certain speed differential. This measure was calculated using a multilevel model and took into account the uncertainty in the estimates of speed differentials.

García et al. (2013) defined a new model of consistency to evaluate the change in speed between straight sections and curves. They called it the Inertial Consistency Index (ICI). It is calculated at the start of a curve, as the difference between the operating speed at that point and the mean operating speed in the preceding kilometer. The new index permits drivers' expectations to be included in the analysis, on the hypothesis that drivers' expectations at a given point on the alignment can be estimated as the average of the operating speed in the preceding segment. The threshholds stablished for the index were: good, when lower than 10 km/h; poor when higher than 20 km/h; and fair otherwise.

A widely used method to evaluate road consistency was developed by Lamm et al. (1999). They presented a design consistency criterion related to operating speed, which include the difference between operating speeds between successives elements (see Criterion I in Table 1).

(Insert Table 1)

The 85th percentile speed difference between successive elements (ΔV_{85}) requires knowing the operating speed V_{85} in each element. Sometimes the operating speed can be measured on the field (Moreno and García, in press) but when observed operating speeds cannot be measured in the field, predicted operating speeds must be used. In the past 50 years, many models for predicting operating speed in curves have been developed. They have also been developed to predict speed on straight sections and to estimate acceleration/deceleration between consecutive elements. A good review of these models can be found in Pérez-Zuriaga et al. (2011a). The speed profile can be built, once curves speed, tangents speed, and deceleration or acceleration rates have been established. Figure 1 represents an operating speed profile along a highway.

(Insert Figure 1)

The length of the tangent between the curves is very important in the speed profile. It is possible to find different cases regarding the distance between curves (see Figure 2):

- Case 1: The tangent length is such that it only allows acceleration or deceleration between curves. (Figure 2 shows deceleration between curves).
- Case 2: The tangent length is such that is possible to achieve an operating speed on tangent, but the length of the straight section is not long enough to reach the maximum operating speed on tangents.
- Case 3: The tangent length is such that allows acceleration up to the maximum operating speed on tangents.
- Case 4: The tangent length is such that it allows acceleration up to the maximum operating speed on tangents and this speed can be maintained.

(Insert Figure 2)

The 85th percentile for the difference in actual driving speeds ($\Delta_{85}V$), which indicates the speed that is not exceeded by 85% of vehicles under free flow conditions (Misaghi 2003) is another variable used to study the differences in speed between successive elements. Hirsh (1987) indicated that $\Delta_{85}V$ is substantially higher than ΔV_{85}. In the same vein, studies in different countries show that ΔV_{85} underestimates drivers' speed reduction between successive elements of the road (McFadden and Elefteriadou 2000; Misaghi and Hassan 2005; Park and Saccomanno 2006; Bella 2007; Nie and Hassan 2007, Castro et al. 2011; Pérez-Zuriaga et al. 2011b). This might also lead to inconsistent designs being considered consistent (Castro et al. 2011).

Models based on operating speed for predicting crashes

There are an important number of models on operating-speed-based measures as predictors of crash rates at horizontal curves on rural two-lane highways (Anderson et al. 1999; Fitzpatrick et al. 2000; Ng and Sayed 2004; Cafiso et al. 2010, Wu et al. 2013). These studies use existing models of speed on curves and tangents, and of deceleration and acceleration rates to determine speed profiles. Anderson et al. (1999) and Fitzpatrick et al. (2000) calculate curve speeds based on a series of speed prediction models developed through regression analysis (Fitzpatrick et al. 1998). Ng and Sayed (2004) calculate curve speeds based on the model developed by Lamm et al. (1999), which modified Morrall and Talarico's (1994) model.

Based on 261 two-lane rural highways sections in New York State, Lamm et al. (1988) studied the 85th percentile speed and crash rates as a function of the degree of curvature. They used this as the basis for obtaining the relationship between speed reduction in the 85th percentile and crash rates. Anderson et al. (1999) and Fitzpatrick et al. (2000) studied the relationship between ΔV_{85} and crashes in 5,287 horizontal curves. Considering a Poisson distribution, they obtained a model that predicted crashes according to the volume of traffic, curve length and speed reduction (see Eq. 1):

$$Y = \exp(-7.1977)\ AADT^{0.9224}\ CL^{0.8419} \exp(0.0662\Delta V_{85}) \quad (1)$$

where:

Y= estimated number of crashes in 3 years for the horizontal curve
AADT= annual average daily traffic (vehicles per day)
CL= curve length (km)

Based on 319 horizontal curves and 511 tangents from Canadian two-lane rural highways, Ng and Sayed (2004) developed eight models relating design consistency measures to safety. Three of those models used ΔV_{85} as independent variable for expected crash frequency per 5 years (see Eqs. 2, 3 and 4).

$$Y_5 = \exp(-3.796)\ CL^{0.8874}\ AADT^{0.5847} \exp(0.04828\Delta V_{85}) \quad (2)$$

$$Y_5 = \exp(-3.369)\ CL^{0.8858}\ AADT^{0.5841} \exp[0.0049(V_{85} - V_d) + 0.0253\Delta V_{85} - 1.177\Delta f_R \quad (3)$$

$$Y_5 = \exp(-2.338)\ L^{1.092}\ AADT^{0.4629} \exp[IC\ (0.022\Delta V_{85} - 1.189\Delta f_R)] \quad (4)$$

where:

Y_5 = estimated number of crashes in 5 years for the horizontal curve
L= section length (km)
V_{85}= 85th percentile speed (km/h)
V_d = design speed (km/h) of a single element

$\Delta f_R = f_R - f_{RD}$

f_R= side friction assumed

f_{RD}= side friction demanded

IC= dummy variable (0 for tangents and 1 for horizontal curves).

Cafiso et al. (2010) calibrated 19 models using a negative binomial distribution to estimate the expected number of crashes on homogeneous sections that could be characterized by constant values of the explanatory variables. They combined variables related to exposure, geometry, consistency and context factors in the 19 models.

Wu et al. (in press) explored the relationship between the number of crashes and design consistency in two highways in Pennsylvania. Design consistency was calculated as the difference between operating speed and inferred design speed. A statistically significant positive association between geometric design consistency and safety was found.

All the models described above are based on the difference between an element's operating speed and the design speed; or the difference between the operating speeds of successive elements. However, the model format, independent variables, and regression coefficients are substantially different from one model to another. This fact, among others, may be due to the difference of driver behavior in different locations. Therefore, most authors (Fitzpatrick et al. 2000; Pérez-Zuriaga et al. 2010; Pérez-Zuriaga et al. 2011a) maintain that a single model cannot be universally accepted, and different models adapted to local circumstances should be used.

The objective of this paper is to obtain crash predictions models for horizontal curves on two-lane rural highways in Spain. Variable speed reduction (ΔV_{85}) plays a role in the models as a consistency criterion. Speed profiles based on Spanish models are used to calculate ΔV_{85}. The results obtained are compared to similar studies conducted on U.S. roads (Fitzpatrick et al. 2000). The models that show the relationship between crashes and the variable $\Delta_{85}V$ are also calculated.

The paper is organized in four main sections. The first section presents an introduction to the main concepts and previous models that relate crash rates to different consistency parameters. The section "Data and Methods" presents the database used and analysis methodology. Finally, a section that presents the results and discussion and a section with the main conclusions of this study are given.

DATA AND METHODS

Data

The study was conducted on 1,748 Km of rural two-lane highways located in the Granada province, in Spain (see Figure 3).

The Granada's rural two-lane highways data were obtained from the Andalucía Regional Government and included roadway inventories, traffic volume and horizontal alignment. The roadways features were reviewed. Thus portions of the roadway were eliminated within small towns or speed zones or in the vicinity of intersections with Stop or signal controls on the main road, intersections with major changes in AADT, and passing or climbing lanes. The remaining highway sections, which total 306 highway sections, were available for analysis. Each section was subdivided into individual horizontal curves and tangents. The final data base included 10,286 horizontal curves with grades between -7% and 7%. The traffic volumes of the study sections ranged from 210 to 8,681 veh/day. Table 2 shows descriptive statistics for the 10,286 curves. This table also shows descriptive statistics for the curves analyzed in Fitzpatrick et al. (2000).

Crash data were obtained from the Spanish General Traffic Crash Directorate (DGT) for a 3-year period (2006-2008). In the 10,286 curves, 261 crashes occurred over the 3-year period with a mean value of 0.025 per curve per year (see Table 2). The crash analysis considered only non-intersection crashes that involved: (1) a single vehicle running off the road; (2) a multiple-vehicle collision between vehicles traveling in opposite directions; or (3) a multiple-vehicle collision between vehicles traveling in the same direction. These are the same crash types that Zegeer et al. (1990) identified as being "over-represented on curves as compared to tangents". All crashes involving parking, turning, or passing maneuvers; animals in the roadway; or bicycles or motorcycles were excluded.

(Insert Table 2)

Determination of the ΔV_{85} and relationship between the ΔV_{85} and crashes

<u>Determination of the ΔV_{85}</u>

To know the difference in ΔV_{85} speeds requires knowing the predicted 85th percentile speed on each tangent and curve, which in turn requires building the speed profile. To build the latter, based on the differences between curves, models of speed on curves and tangents, and of deceleration and acceleration rates were selected (see Figure 2).

Five speed profiles were built for this work, based on combinations of the models of speeds on curves, speed on tangents, and various accelerations and decelerations rates.

Speed on curves

The introduction highlights the importance of using speed-prediction models that are calibrated according to local conditions. Consequently, in this paper, we use the models proposed by Castro et al. (2008) (see Eq 5) and Camacho-Torregrosa et al. (in press) (see Eq 6 and 7) to calculate the speed on curves for the various profiles, adjusted for horizontal curves for two-lane rural highways in Spain.

$$V_{85} = 120.16 - 5{,}596.72/R \quad (5)$$

$$V_{85} = 97.4254 - 3{,}310.94/R \text{ for } 400 \text{ m} < R \leq 950 \text{ m} \quad (6)$$

$$V_{85} = 102.048 - 3{,}990.26/R \text{ for } 70 \text{ m} < R \leq 400 \text{ m} \quad (7)$$

where:

R= horizontal curve radius (m).

Although Castro et al. (2008) did not specify the range for Eq 5, they told us that their equation was valid for radii greater than 70 m and less than 950 m.

For radiuses lower than 70 m and higher than 950 m in the above three equations, we took the specific speeds for curves with those radiuses outlined in the Spanish standard Norma 3.1-IC de Trazado (Ministerio de Fomento 1999).

Speed on tangents

Speed on tangents is in accordance with the curve speed model:

- If the curve speed used is the one defined in equation 5, then the tangent speed used is that corresponding to infinite radius in equation 5 (Castro et al. 2008).
- If the curve speed used is that defined in equations 6 and 7, the tangent speed value taken is 110 km/h (Camacho-Torregrosa et al. in press).

Acceleration and deceleration rate

The rates of deceleration and acceleration applied between tangent and curve in order to change from tangent speed to curve speed were also different in the various profiles.

Three different approaches for the acceleration and deceleration rate have been used:

- a constant acceleration and deceleration rate of 0.85 m/s^2 (Krammes et al 1995; Transportation Association of Canada 1999);
- a variable acceleration and deceleration rates proposed by Fitzpatrick et al (2000) (see Table 3); and
- a variable acceleration (see Eq 8) and deceleration rates (see Eq 9) proposed by Camacho-Torregrosa et al (in press) and Pérez-Zuriaga et al (2010) respectively. These models were calibrated on Spanish roads.

$$a_{85} = 0.41706 + 65.93588/R \quad (8)$$

$$d_{85} = 0.313 + 114.436/R \quad (9)$$

The value of R^2 in Eq 8 is not available and the value of R^2 in the Eq 9 was 66.5%.

(Insert Table 3)

Table 3 shows how the acceleration rate becomes lower with a larger radius. This is because the greater the curve's radius, the greater the speed that can be reached on it. Therefore, the acceleration to reach the desired speed on the next tangent will be lower than it would be if the speed on the curve we start out with is lower.

Table 4 shows the profiles created with different combinations of curve speeds, tangent speeds, and acceleration and deceleration rates. These profiles will be used to evaluate the speed reductions between successive tangents and horizontal curves and between successive curves (i.e., tangent/curve, curve/tangent, or curve/curve). Figure 4 shows a graphical representation of the five speed profiles on a section of a highway.

(Insert Table 4)

(Insert Figure 4)

The speed profiles are similar (see Figure 4), although profiles 1 and 2 provide higher speeds than the other profiles (about 12 km/h). Because the shape of the profiles is very similar, the differences in 85th percentile speeds are also very similar.

Although the profiles are similar in shape, profiles 1 and 2, built with the curve speeds of Castro et al (2008), provide higher speeds than profiles 3, 4 and 5, which were built with the curve speeds of Camacho-Torregrosa et al. (in press). It can also be seen that the constant acceleration and deceleration of 0.85 m/s^2 in profiles 1 and 3 give very high changes in speed (higher peaks) than the accelerations and decelerations that vary according to the radiuses (Table 3) in profiles 2 and 4. Finally, the acceleration and deceleration calibrated for Spanish roads in profile 5 (Eqs 8 and 9) give intermediate speeds between the constant acceleration and the ones in Table 3.

Relationship between the ΔV_{85} and crashes

For the analysis of the relationship between the ΔV_{85} and crashes, this paper followed the same methodology used for the Interactive Highway Safety Design Model (IHSDM) developed by the Federal Highway Administration (Fitzpatrick et al. 2000). This model is presented in Eq.1.

In the IHDSM, an analysis is made of speed reduction as a design consistency criterion for horizontal curves in two-lane rural highway in Washington State. Using a Poisson regression models the relationship between crash frequencies and exposure, curve geometries, and speed-reduction variables was investigated. The variables included in the Poisson models were crashes for 3 years as a dependent variable and AADT (veh/day), horizontal length (km) and speed reduction (km/h) as independent variables.

In this paper, the relationship between crash frequencies and selected variables (AADT, horizontal curve length and speed reduction) was investigated using loglinear regression models and Poisson or Negative Binomial (NB) distribution. The Poisson distribution is an appropriate choice since crash frequencies are: (1) integers, (2) relatively small numbers, and (3) necessarily non-negative. However, a basic assumption underlying the use of the Poisson distribution is that its mean and variance are equal. When this assumption is substantially violated, the NB distribution can provide an improvement over the Poisson distribution (Lord and Mannering (2010).

The zero-inflated model is applied when the observed data are characterized by large zero densities. According to Lord and Mannering (2010), the zero-inflated model (both for the Poisson and negative binomial models) has been popular among transportation safety analysts. Despite its broad applicability to a variety of situations where the observed data are characterized by large zero densities, others have criticized the application of this model in highway safety. For instance, Lord et al. (2005, 2007) argued that, because the zero or safe state has a long-term mean equal to zero, this model cannot properly reflect the crash-data generating process.

Like in the previous study conducted by Fitzpatrick et al. (2000), AADT and curve length are included in the model and their coefficients are estimated separately. The natural logarithms of AADT and curve length, rather than their untransformed values, are used in modeling. The functional form for this analysis is a multiplicative model in the form:

$$Y = \exp(\beta_0)\ AADT^{\beta 1}\ CL^{\beta 2} \exp(\beta_3 \Delta V_{85}) \tag{10}$$

The ΔV_{85} was calculated for each of the five speed profiles in Table 4, to obtain five crash prediction models, depending on the speed profile used to obtain the ΔV_{85}.

Determining the $\Delta_{85}V$ and the relationship between the $\Delta_{85}V$ and crashes

Determination of the $\Delta_{85}V$

Three models were used to determine the $\Delta_{85}V$ in the 10,286 study curves.

The first one is the model proposed by Pérez-Zuriaga et al. (2011b), which was calibrated for horizontal curves on two-lane rural highways in Spain (see Eq. 11)

$$\Delta_{85}V = 10.005 + 1299.733/R \tag{11}$$

where:
R= horizontal curve radius (m).

The second one is the model proposed by Bella (2008) (see Eq.12), in which the variable the 85th percentile Maximum Reduction in Speed (85MSR) is defined as the 85th percentile of the maximum reductions in individual speed along a section made up of the last 200 meters of a straight section approach and the mid-point of a curve. Bella (2008) used driving simulators to make a comparative study of the two variables (85MSR and $\Delta_{85}V$). Eq. 12 shows the relationship between the two variables. The expression proposed by Bella (2007) was used to calculate the 85MSR (see Eq. 13).

$$\Delta_{85}V = (85MSR - 6.35)/1.08 \tag{12}$$

$$85MSR = -0.198 + 0.037L_{at} + 7929.37/R \tag{13}$$

where:
L_{at}= length of approach tangent (m)

The third model used for the analysis of $\Delta_{85}V$ was proposed by Nie and Hassan (2007) (see Eq. 14), based on the curvature change rate (CCR) (gon/km):

$$\Delta_{85}V = -4.540 + 0.088CCR \tag{14}$$

Relationship between the $\Delta_{85}V$ and crashes

To carry out the analysis of the variable $\Delta_{85}V$ with crashes, the variable $\Delta_{85}V$ was calculated according to the models in Eqs 11, 12 and 14. According to these equations the $\Delta_{85}V$ depends of the geometric features of the curves as they depend of R or of the CCR. The crashes were the same that are mentioned in this section. The expression of the model used to calculate the estimated number of crashes was the same as the one used for the variable ΔV_{85}:

$$Y = \exp(\gamma_0)\ AADT^{\gamma 1}\ CL^{\gamma 2}\ \exp(\gamma_3 \Delta_{85} V) \tag{15}$$

All the models were fitted using STATA statistical package (StataCorp 2011).

RESULTS AND DISCUSSION

Following the process described in the methodology, 5 speed profiles were built in each of the 306 study highway sections, using the combinations of curve speeds, tangent speeds and acceleration and deceleration rates shown in Table 4. The speed reductions between successive tangents and horizontal curves and between successive curves for each of the 10,286 curves can be obtained from each profile (see Table 5).

(Insert Table 5)

Relationship between ΔV_{85} and crashes

On the 10,286 curves, 261 crashes occurred over the 3-year period, with an average value of 0.025 per curve per year. However, a large proportion of the curves (99.79 percent) experienced one or no crash during the 3-year period. The distribution of these crashes is shown in Table 6.

(Insert Table 6)

The 10,286 individual horizontal curves were classified as good, fair, and poor with respect to design safety using the criterion I in Table 1. Table 7 shows a summary of the crash frequencies, exposure (vehicle-kilometers of travel), and crash rates for these 10,286 curves and for each one of the speed profiles described in Table 4. The results from the study by Fitzpatrick et al. (2000) have been given to facilitate a comparison with ours.

(Insert Table 7)

It can be observed that the results were very similar, regardless of the operating-speed profile used. In all the cases the average crash rate is highest for the horizontal curves in the poor category and lowest for the horizontal curves in the good category. In the analysis carried out by Fitzpatrick et al. (2000), the average crash rate is also highest for poor category and lowest for good category. However, the mean crash rate value (crashes/million veh-km) is higher in the U.S. study (0.52) than in the Spanish study (0.17).

The relationship between crash frequencies and selected variables (AADT, horizontal curve length and ΔV_{85}) was investigated using loglinear regression models and Poisson, and not a NB distribution because the variance was 0.031, a value almost equal to the mean.

In addition, after regressions, goodness-of-fit tests of the models were carried out. If the test statistic is significant (p-value equal 0) indicates that a Poisson model is inappropriate and Negative Binomial regression is better. In the case of study, the test statistic is not significant (p-value equal 1) for all profiles. Therefore, the test leads to the same conclusion: the Poisson regression model is appropriate.

Despite some authors criticized the applications of the zero-inflated model (Lord et al., 2005, 2007), in this study, each of the five Spanish models have been calculated with a Poisson distribution and with a Zero inflated model for the Poisson distribution to compare them. According to Lord et al (2005, 2007) and because the results showed the Poisson distribution was better than Zero inflated model in the profiles 1, 3 and 5 (and in the profiles 2 and 4 the differences between using one model or another were minimal), a Poisson distribution has been considered in this study.

Table 8 shows the results of the models fit following Eq.10. For the purposes of comparison, we have included the results of the model adjusted by Fitzpatrick et al. (2000) for development of the IHSDM by the Federal Highway Administration. For the model proposed in Eq. 10 (called Model 1 from this moment), all three variables (AADT, CL and ΔV_{85}) were significant at the 95-percent confidence level for all profiles except speed profile 3 (it is significant at 90-percent confidence interval) and show the direction of correlation as expected. In order of importance of their contribution to explaining the variability in the crash data, as measured by their χ^2 (Pearson value), the three parameters rank as follows: log (AADT), log (CL) and ΔV_{85} (see Table 8). These results are similar to those obtained by Fitzpatrick et al. (2000).

The overall fit of a Poisson model can be assessed using the following goodness-of-fit criteria: the dispersion parameter (a measure of over- or under-dispersion of the data, which, under ideal conditions, should be close to 1); the Pearson chi-square (another measure similar to the dispersion parameter, which should also be close to 1; generally, a value between 0.8 and 1.2 is an indication that the model can be assumed to be appropriate in modeling the data); the ordinary multiple correlation coefficient (R^2) and the Freeman-Tukey correlation coefficient (R^2_{FT}) (Moses and Holland 2010), each correlation with a maximum of 100 percent. All these criteria are included in Table 8.

For Model 1, in all profiles, the dispersion parameter ranged from the lowest value (0.927) obtained for Profile 3 to the highest value (0.955) for Profile 4. The dispersion parameter value close to 1 is an indication that the model can be considered appropriate in modeling the data. All values are close to the 0.830 obtained in the model by Fitzpatrick et al. (2000). The values of R^2 ranged between 15.89% (Profile 3) and 16.18% (Profile 4). R^2_{FT} is between 15.78% (Profile 3 and Profile 5) and 15.71% (Profile 4). All these are also close, although lower than those obtained by Fitzpatrick et al. (2000) (R^2=19.5% and R^2_{FT}=17.9%). The coefficient between the chi-square value for a 95% confidence level and the sample's chi-square value gave values between 1.071 (Profile 4) and 1.103 (Profiles 3), which are also close to the 1.21 obtained by Fitzpatrick et al. (2000). Although the values of R^2 obtained in this research and the ones obtained by Fitzpatrick et al. (2000) are relatively low, Newman and Newman (2000) suggest that a low R^2 may not necessarily be a wrong path. It may only be a partial explanation of the dependent variable and further research will improve it by adding additional predictor variables. Moreover, Gujarati (2004) and Wooldridge (2009) suggest that, although R^2 may be low, if one or more regression coefficients are statistically significant, then, the relationship between predictors and the response may be important, even though it may not explain a large amount of variation in the response. They suggest that a good relationship between explanatory variables and the dependent variable does not depend directly on the size of R^2. In this study, the statistics of the overall fit test of the model are all significant. This shows the good fit of the models. In our opinion, the R^2 values obtained are due to the fact that crashes are influenced by other variables (e.g alcohol, driver distraction; excessive speed in driving or conditions atmospheric). Such variables are not considered in this model because it focuses on road and traffic features.

(Insert Table 8)

The comparison of the results of this research conducted on Spanish roads and U.S. roads (Fitzpatrick et al. 2000) shows that in both cases, the three variables are important for the crashes. By order of the importance of their contribution to explaining crashes, the three variables rank as follows: log (AADT), log (CL) and ΔV_{85}.

The models obtained with the five speed profiles are very similar to each other. The statistical equality was confirmed with least significant difference (LSD) intervals (see Figure 5). As the intervals overlap, the population means are not significantly different from each other at the 95% confidence level.

(Insert Figure 5)

This result was expected because the speed profiles are also very similar (see Figure 4).

The U.S. model has been applied to Spanish data. The U.S. model was compared to each Spanish model (built with each profile) (see Figure 6). The statistical difference was confirmed with least significant difference (LSD) intervals.

(Insert Figure 6)

As the intervals do not overlap, the population means are significantly different (p-value equal 0) from each other at the 95% confidence level. The Spanish model and the U.S. model are significantly different. The U.S. model predicts more crashes than any of the 5 Spanish models. These results are consistent with the results in Table 7, which shows that the crash rate in the U.S. study is higher than in the Spanish study.

In our opinion, the higher number of crashes estimated for the U.S. than for Spain is due, on the one hand, to the fact that the AADT on the roads in the U.S. study nearly doubles the AADT on the roads in the Spanish study (2,283 veh/day vs 1,160 veh/day; see Table 2). This can be translated to a higher number of crashes predicted with the U.S. model. Some authors (Khan et al. 2013, Quddus 2013) have already pointed out the fact. Khan et al. (2013) performed a regression tree analysis to explore horizontal curve safety and the results show that there was a marked increase in the number of crashes on horizontal curves, with traffic volume greater than approximately 1,300 vehicles per day. Quddus (2013), when analyzing the relationship between mean speeds, speed variations and crash rates, found that speed variation was statistically and positively associated with crash rates. He also demonstrated that the effects of all other factors on crash rates were found to be consistent with existing studies. For instance, a 1% increase in AADT is related to a 0.5% increase in killed and serious injury crash rates. On the other hand, it is possible that the Spanish model predicts fewer crashes than the U.S. model, owing to the roads of our study have more curves. Some researchers defend that although individual horizontal curves may be hazardous, frequent horizontal curves can have a protective effect, because the driver may be more cautious (Milton and Mannering 1998, Haynes et al. 2007; Wang et al. 2009; Jones et al. 2012). Findley et al. (2012) showed that the more closely spaced curves have fewer predicted collisions than those curves which are more distant to each other. The authors state that the findings are consistent with the concept of driver expectations used by highway designers which asserts that a road that violates a driver's expectations will be likely to have more crashes than a road that does not violate those expectations. When analyzing the relation between crashes and curve features in 10 countries, Elvik (2013b) found that crashes diminish when the distance between curves is shorter.

Relationship between $\Delta_{85}V$ and crashes

The goodness-of-fit-tests, after regression, show that the Poisson regression model is also appropriate for Eq.15.

Table 9 shows the results of the models fit according to Eq.15 (hereinafter referred to as Model 2) and using the three different approaches for $\Delta_{85}V$ (see Eqs.11, 12 and 14).

(Insert Table 9)

The three models used in Table 9 give results that are very similar to each other, and to the results shown in Table 8. They show the direction of correlation as expected. The order of importance in the variables to explain crash data is confirmed: log (AADT), log (CL) and $\Delta_{85}V$ (see Pearson values in Table 9). The statistical equality between the three models created with the variable $\Delta_{85}V$ was confirmed with least significant difference (LSD) intervals (see Figure 7). As the intervals overlap, the population means are not significantly different from each other at the 95% confidence level. Figure 7 includes the profile created with the variable ΔV_{85} (profile 1) to show that there are no statistically significant differences between the models created with the ΔV_{85} and the models created with the variable $\Delta_{85}V$.

(Insert Figure 7)

The adjustment obtained for the model considering the variable ΔV_{85} and the adjustment obtained for the variable $\Delta_{85}V$ (Models 1 and 2) was so similar that it is impossible to highlight one more than the other to explain the variation in crash data. The similarity between the two models may be because although the speeds calculated with $\Delta_{85}V$ are higher than those calculated with the variable ΔV_{85}, the speed differences are similar in both cases, resulting little differences in the models.

In both relationships, the speed reduction effect is highly significant (see Tables 8 and 9) because the significance level in both cases was less than 0.05, except for speed profile 3 in Table 8, which was less than 0.07. The direction of both relationships indicates that the greater the speed reduction experienced by motorists on a horizontal curve, the greater the curve's crash experience. This also explains why speed reduction is one of the main measures of design consistency.

CONCLUSIONS

In this paper a Poisson distribution was used to study the relationship between crashes and ΔV_{85}, AADT and curve length (CL) in horizontal curves on two-lane rural highways, with grades between -7% and 7%. Five Spanish models were built in which the AADT and CL was the same for all, but the ΔV_{85} was calculated for five different speed profiles that were built using several combinations of Spanish models. The results obtained with the five Spanish models are compared with results obtained in similar study conducted on U.S two-lane rural highways (Fitzpatrick et al. 2000).

In the U.S model and the 5 Spanish models, the three independents variables considered are important for the crashes, and by order of importance of their contribution to explaining crashes, the three variables rank as follows: log (AADT), log (CL) and ΔV_{85}. The low adjustment values in the Spanish models and in the U.S. models indicating that a large proportion in the variation in crash data is not accounted for by the Poisson model, which confirms that crashes are influenced by other variables, such as those related to driver behavior or vehicle, which this study does not analyze.

The 5 Spanish models show no statistically significant differences between them. The similarity in the different models obtained using the different speed profiles is because the shape of the profiles is very similar (although moved upwards in the profiles 1 and 2), which provides similar difference in 85th percentile speeds.

When the U.S. model is applied to the Spanish data, the U.S. model shows statistically significant differences with any of the 5 Spanish models. The model designed by Fitzpatrick et al. (2000) predicts more crashes than the Spanish models. In our opinion, this is because the AADT on U.S. roads is higher than the AADT in Spain, which would mean a higher crashes prediction in the U.S model. On the other hand, the Spanish roads analyzed have more curves. Some researchers defend that although individual horizontal curve might be hazardous, frequent horizontal curves may have a protective effect, because drivers would be more cautious. The latter aspect would be worth studying in further depth, so we suggest it as a future line of research.

In addition other models were adjusted to take AADT and curve length into consideration, after replacing the independent variable ΔV_{85} with the $\Delta_{85}V$. The models calibrated with the variable $\Delta_{85}V$ show no statistically significant differences with any of the models calculated using the variable ΔV_{85}. The similarity between the two models may be because although the speeds calculated with $\Delta_{85}V$ are higher than those calculated with the variable ΔV_{85}, the speed differences are similar in both cases, resulting little differences in the models. Model adjusted with the independent variable $\Delta_{85}V$ show the direction of correlation as expected, with being log (AADT) the most important variable and $\Delta_{85}V$ the least important to explaining crash.

The main limitation of this paper is the fact that speed models adapted to local conditions are used, not actual operating speeds. That is why we consider future lines of study to calculate actual speed profiles in some of the study roads and compare them with the theoretical profiles obtained in this study.

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Table 1. Consistency Criterion proposed by Lamm

Consistency rating	Criterion I (km/h)
Good	$\Delta V_{85} \leq 10$
Fair	$10 < \Delta V_{85} \leq 20$
Poor	$\Delta V_{85} > 20$

$\Delta V_{85} = |V_{85i}-V_{85i+1}|$; difference in 85th percentile speed between successive geometric elements (km/h)

Table 2. Descriptive statistics for 10,286 horizontal curves

	Spanish roads			Fitzpatrick et al. (2000)		
Parameter	Minimum	Mean	Maximum	Minimum	Mean	Maximum
Number of accidents in 3 years	0	0.025	4	0	0.33	11
AADT (veh/day)	210	1,160	8,681	222	2,283	18,005
Horizontal curve length (Km)	0.015	0.105	1.094	0.016	0.238	2.977
Exposure (million veh-km)	0.005	0.149	4.700	0.006	0.638	19.777
Horizontal curve radius (m)	16	230.1	2,825	19.5	860.8	15,250

Table 3. Deceleration and acceleration rates for speed profile in horizontal curves with grade between -9% and 9% proposed by Fitzpatrick

Deceleration Rate, d (m/s^2)		Acceleration Rate, a (m/s^2)	
Radius, R (m)	d	Radius, R (m)	a
$R \geq 436$	0.00	$R > 875$	0.00
		$436 < R \leq 875$	0.21
$175 \leq R < 436$	$\lvert 0.6794 - 295.14/R \rvert$	$250 < R \leq 436$	0.43
$R < 175$	1.00	$175 < R \leq 250$	0.54

Table 4. Speed profiles for different speeds models and acceleration and deceleration rates

	Curve Speed (km/h)	Tangent Speed (km/h)	Acceleration (m/s^2)	Deceleration (m/s^2)
Profile 1	Eq 5	InfiniteRadius in Eq 5	0.85	0.85
Profile 2	Eq 5	InfiniteRadius in Eq 5	Table 3	Table 3
Profile 3	Eqs 6 and 7	110	0.85	0.85
Profile 4	Eqs 6 and 7	110	Table 3	Table 3
Profile 5	Eqs 6 and 7	110	Eq 8	Eq 9

Table 5. Descriptive statistics for speed reductions

Speed reduction (km/h)	Minimum	Mean	Maximum
Profile 1	0	15.04	60.16
Profile 2	0	14.71	60.16
Profile 3	0	10.92	50
Profile 4	0	10.03	50
Profile 5	0	10.76	50
Profile 6	0	10.38	50
Fitzpatrick et al. (2000)	0	3.91	32.4

Table 6. Crash frequency distribution for horizontal curves

Number of crash in 3 years	Number of curves	Percentage of curves
0	10,052	97.73%
1	212	2.06%
2	19	0.18%
3	1	0.01%
4	2	0.02%

Table 7. Crash rates at horizontal curves by design safety level

Design Safety Level	Speed profile 1	Speed profile 2	Speed profile 3	Speed profile 4	Speed profile 5	Fitzpatrick et al. (2000)
			3-Year Accident Frequency			
Good	118	124	118	152	119	1,483
Fair	62	54	86	60	87	217
Poor	81	83	57	49	55	47
Combined	261	261	261	261	261	1,747
			Exposure (million veh-km)			
Good	832	890	764	1,030	779	3,206
Fair	339	267	543	285	514	150.46
Poor	359	373	223	215	237	17.05
Combined	1,530	1,530	1,530	1,530	1,530	3,374
			Crash Rates (crashes/million veh-km)			
Good	0.14	0.14	0.15	0.15	0.15	0.46
Fair	0.18	0.20	0.16	0.21	0.17	1.44
Poor	0.23	0.22	0.26	0.23	0.23	2.76
Combined	0.17	0.17	0.17	0.17	0.17	0.52

Table 8. Relationship between AADT, CL and ΔV_{85} and road safety

MODEL FORM: $Y = \exp(\beta 0) AADT^{\beta 1} CL^{\beta 2} \exp(\beta 3 \Delta V_{85})$								
	β_0	β_1	β_2	β_3	Dispersion Parameter	$\chi^2_{0.05}/\chi^2$	R^2	R^2_{FT}
Profile 1	-9.8340	1.1326	0.9633	0.0121	0.9420	1.086	15.98%	15.75%
		$\chi2$=178.34	$\chi2$=126.28	$\chi2$=5.57				
		RSE=0.0848	RSE=0.0857	RSE=0.0051				
p-value	<0.0001	<0.0001	<0.0001	0.018				
Profile 2	-9.8012	1.1325	0.9783	0.0125	0.9446	1.083	16.05%	15.74%
		$\chi2$=180.39	$\chi2$=126.42	$\chi2$=7.43				
		RSE=0.0843	RSE=0.0870	RSE=0.0046				
p-value	<0.0001	<0.0001	<0.0001	0.006				
Profile 3	-9.8379	1.1209	0.9165	0.0150	0.9274	1.103	15.89%	15.78%
		$\chi2$=178.63	$\chi2$=121.32	$\chi2$=3.29				
		RSE=0.0838	RSE=0.0832	RSE=0.0083				
p-value	<0.0001	<0.0001	<0.0001	0.070				
Profile 4	-9.8622	1.1388	0.9889	0.0216	0.9554	1.071	16.18%	15.71%
		$\chi2$=179.92	$\chi2$=125.46	$\chi2$=9.79				
		RSE=0.0849	RSE=0.0883	RSE=0.0069				
p-value	<0.0001	<0.0001	<0.0001	0.002				
Profile 5	-9.8417	1.1226	0.9292	0.0161	0.9284	1.090	15.92%	15.78%
		$\chi2$=180.38	$\chi2$=123.76	$\chi2$=4.06				
		RSE=0.0836	RSE=0.0835	RSE=0.0080				
p-value	<0.0001	<0.0001	<0.0001	0.044				
Fitzpatrick et al. (2000)	-7.1977	0.9224	0.8419	0.0662	0.830	1.21	19.5%	17.9%
		$\chi2$=863	$\chi2$=638	$\chi2$=200				
p-value	<0.0001	<0.0001	<0.0001	<0.0001				

$\chi2$: Pearson value
RSE: Robust Standar Error

Table 9. Relationship between AADT, CL and $\Delta_{85}V$ and road safety

MODEL FORM: $Y = \exp(\gamma 0) AADT^{\gamma 1} CL^{\gamma 2} \exp(\gamma 3 \Delta_{85}V)$								
$\Delta_{85}V$	$\gamma 0$	$\gamma 1$	$\gamma 2$	$\gamma 3$	Dispersion Parameter	$\chi^2_{0.05}/\chi^2$	R^2	R^2_{FT}
Pérez-Zuriaga et al. 2011a	-9.9017	1.1309	0.9957	0.0154	0.9406	1.088	15.98%	15.75%
		$\chi2$=190.66	$\chi2$=116.36	$\chi2$=9.60				
		RSE=0.0819	RSE=0.0923	RSE=0.0050				
p-value	<0.0001	<0.0001	<0.0001	0.002				
Bella, 2008	-9.6777	1.1231	0.9593	0.0015	0.9248	1.106	15.84%	15.79%
		$\chi2$=185.57	$\chi2$=122.64	$\chi2$=6.24				
		RSE=0.0824	RSE=0.0866	RSE=0.0006				
p-value	<0.0001	<0.0001	<0.0001	0.012				
Nie and Hassan, 2007	-9.7148	1.1287	0.9855	0.0036	0.9347	1.094	15.97%	15.76%
		$\chi2$=188.93	$\chi2$=118.79	$\chi2$=9.83				
		RSE=0.0821	RSE=0.0904	RSE=0.0012				
p-value	<0.0001	<0.0001	<0.0001	0.002				

$\chi2$: Pearson value
RSE: Robust Standar Error

Figure 1
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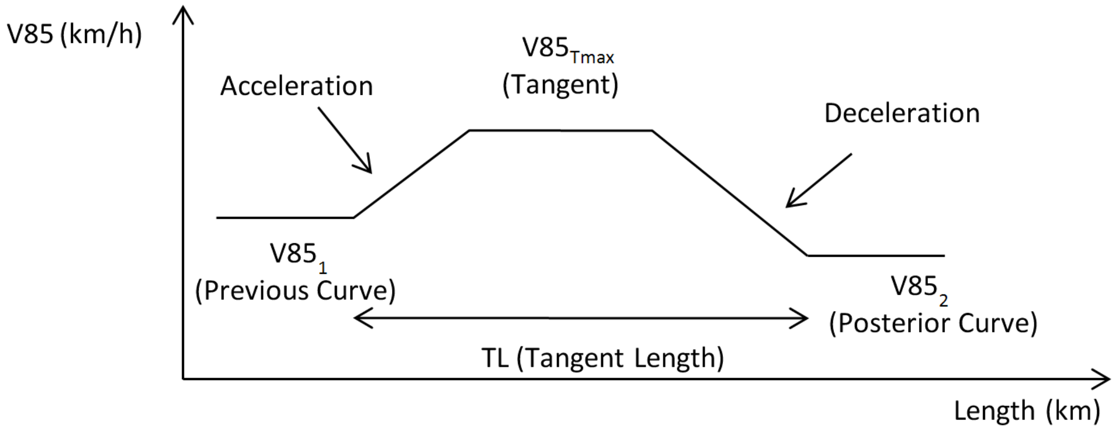
Figure 1. Speed Profile



Note:
$V85_1$: 85th percentile speed on Previous Curve
$V85_2$: 85th percentile speed on Posterior Curve
$V85_{Tmax}$: maximum operating speed on Tangents
TL: existing Tangent Length between two successive curves

Figure 2
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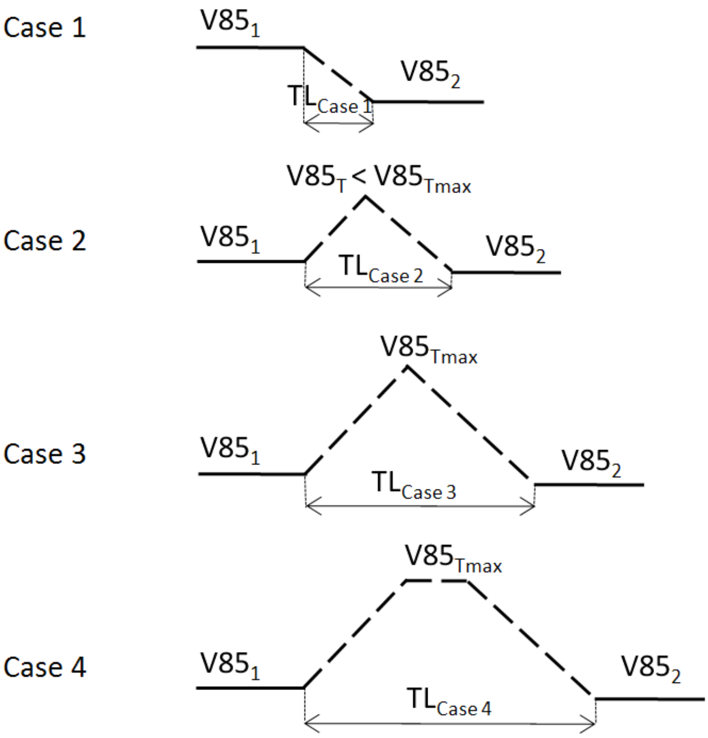
Figure 2. Different cases for speed profiles



Note:
$V85_1$: 85th percentile speed on Previous Curve
$V85_2$: 85th percentile speed on Posterior Curve
$V85_{Tmax}$: maximum operating speed on Tangents
$V85_T$: Operating speed on tangents ($V85_T$ can reach to a maximum of $V85_{Tmax}$)
$TL_{Case\ i}$: existing Tangent Length between two successive curves in Case i

Figure 3
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Figure 3. Location of Granada in Spain

Figure 4
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Figure 4. Graphical representation of the five speed profiles on a section of a highway

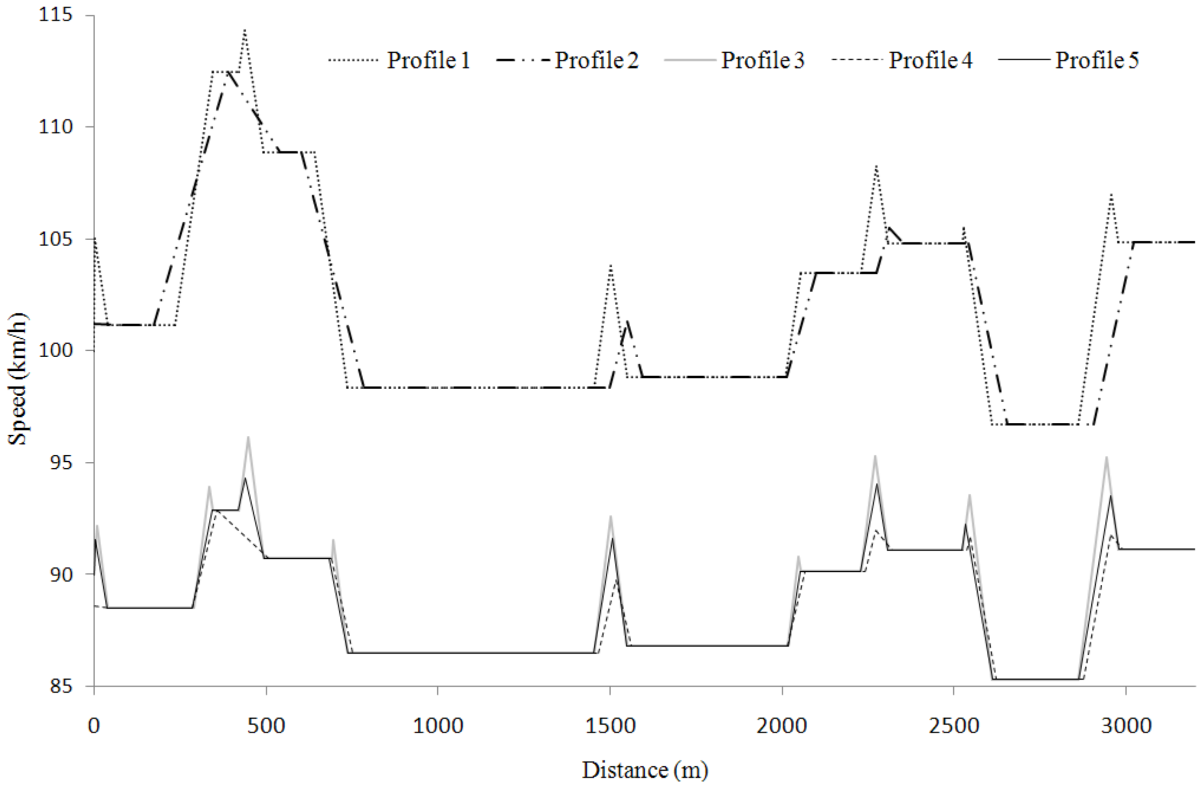

Figure 5
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Figure 5. LSD intervals (95.0 percentage) for the different profiles built with ΔV_{85}

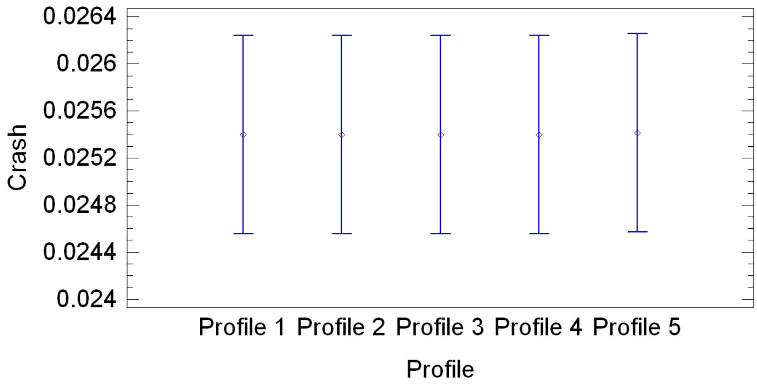

Figure 6. LSD intervals (95.0 percentage) for the U.S. model and the five Spanish profiles

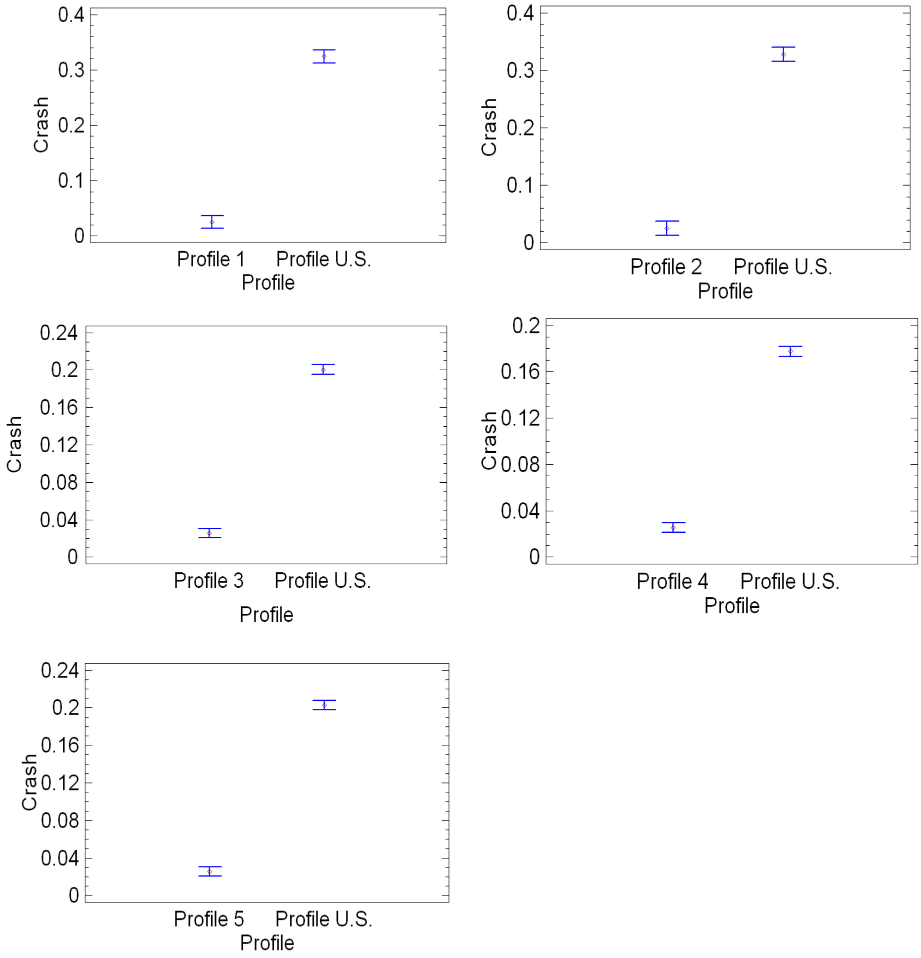

Figure 7
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Figure 7. LSD intervals (95.0 percentage) for the different models built with $\Delta_{85}V$