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EMPIRICAL MODELING OF THE RELATIONSHIP BETWEEN DECISION SIGHT DISTANCE AND STOPPING SIGHT DISTANCE BASED ON AASHTO

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Abstract:

The paper introduces implementation of highways' stopping sight distance (SSD) and decision sight distance (DSD) based on AASHTO modeling assumptions. SSD characterizes the necessary distance for highway vehicles to stop safely in front from an obstacle. SSD is a function of vehicle speed, perception reaction time, deceleration rate, and grade based on AASHTO and most highway design international guidelines. The deceleration rate which is assumed constant (3.4 m/sec^2) based on AASHTO 2011 is generally controlled by the friction coefficient depending on the road surface conditions. A driver's demanded deceleration rate may not exceed the range of friction coefficient according to various pavement conditions. Although SSD is generally sufficient to allow skilled and alert drivers to the stop their vehicles under regular situations, this distance is insufficient when information is difficult to comprehend. A DSD should be provided in highways geometric design when the driver is required to detect an unexpected or difficult to perceive information source. Interchanges (specifically exit ramps) and intersections, and required changing in driver direction of travel, changes in the basic cross section such as toll plaza, lane drop, are typical scenarios where driver needs DSD in the safety manner. The introduction of the two sight distance types (SSD and DSD) is a perquisite for empirical modeling of the relationship between DSD and SSD. The modeling refers to DSD for rural highways, suburban roads, and urban roads based on AASHTO models. Specifically the paper covers DSD three avoidance maneuver types of stopping (types A, A1, B) and three maneuver types of speed, path, and direction changing (types C,D, E) for the three roadway categories. The major parameters that control these avoidance types are pre-maneuver times, and pre-maneuver plus maneuver times. The empirical relationship proposed in this study simplifies the process of evaluating the decision sight distance based on stopping sight distance record, based on AASHTO models, without the need of strenuous estimation of the DSD model maneuver and deceleration parameters. Such a simplified correlation has not been found in the literature except a rough approximation documented in the British highway design guidelines.

Key words:
sight distance, decision, stopping, maneuver, speed, decelerartion

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1. Introduction

A major purpose in highway geometric design is to ensure safe driving in the highway system. Safe highways must be designed in order to provide drivers a sufficient distance of clear vision ahead and avoid colliding unexpected objects either by stopping, or by passing slower vehicle ahead by changing the lane without danger. Sight distance is a fundamental issue in highway design policy which directly affects the highway alignment, specifically vertical curves, and horizontal curves, in order to maintain highway safety for the drivers. Sight distance is therefore, the length of highway ahead which is visible to the driver. When this distance is not long enough to permit passing an overtaken vehicle or safe maneuvering upon an obstruction it is termed stopping sight distance. Sufficient sight distance must be provided to allow for high percentage of drivers of all skills and training levels to stop or maneuver around obstacles on the roadway surface or make safe lane changing or turns depending on the highway design policy.

The first part of this paper overviews the details of two sight distance types based on AASHTO highway design policy (AASHTO 2004, 2011): stopping sight distance (SSD), and decision sight distance (DSD). This overview covers the parameters of perception reaction time (PRT) and deceleration coefficient required for SSD, and additional parameters of pre maneuver time, and maneuver time, required for several DSD avoidance maneuver types. The assessed sight distance computed values function as an input for the major purpose of this study which is calibrating a model which directly (empirically) formulates the relationship between DSD and SSD. Such a simplified correlation has not been found in the literature except a rough approximation documented in the British highway design guidelines (DMRB 1993, NRA 2007). The advantage of this empirical relationship between DSD and SSD is that after computing SSD we can obtain the DSD length of certain avoidance type and highway category (e.g. rural, urban, suburban) without the need of estimating its relevant maneuver time parameters.

1.1. Design speed and its design controls affecting SSD and DSD

The **design speed** (Vd) is a selected speed which is used to determine the various geometric design features of the roadway (AASHTO 2011). The implementation of design speed is performed by determining the threshold values of the highway alignment features such as minimum stopping sight distance, minimum radius along horizontal curve and vertical curve, and maximum grade, and the coordination between them. The design speed is therefore the safest speed that is determined for the highway geometric design and its geometric components, which influence vehicle operation. The selection of design speed necessitates balanced geometric design and measures of design consistency.

The **operating speed** is the speed at which drivers are observed operating their vehicles during free flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric feature (AASHTO 2011).

Leisch and Leisch (1977) correlated the design speed with operating speed by defining the design speed as a representative potential operating speed that is determined by the design and correlation of the physical (geometric) features of a highway.

The **posted speed** (maximum speed limit) is a speed limitation for drivers introduced due to safety, economy, traffic control. and government regulatory policy. It is applied for encouraging drivers to travel at an appropriate speed for surrounding conditions (TAC 1999).

The target speed is the desirable travel speed in the defined highway category. The goal is for most vehicles (90%) in the traffic stream to be able to travel at such a speed during free-flow conditions in a specific highway category. In order to maintain design consistency, it is vital that maximum speed limit and target speed would be similar. Israeli policy indicates that in order to provide a reasonable safety margin (similarly to other civil engineering disciplines and in accordance with international guidelines), the value of the design speed is in practice 10 km/hour faster than the value of the target speed on the interurban network.

Every highway type incorporates the maximum design speed and a reduced range of design speeds according to certain sensitivity constraints (e.g. topography, environment, right of way, urban issues, etc.). A reduction of the design speed should be accompanied by a reduction in the posted speed limit (or target speed). This principal (which is recommended by Israel highway design guidelines regarding the relationship between the design speed and posted speed limit) is based on literature review from the United States and Europe (Bassan 2016). Empirical estimations performed in the U.S. have shown that the approximate difference between the posted speed limit (which could represent the target speed) and the operating speed (which could represent the design speed) is 10 km/hour for the higher speed range (Fitzpatrick et al. 2005). In Canada, the differences are larger due to considerably high upper range of design speeds in most highway types. According to OECD (2006), the required design speed depends on the function of the road and, hence, on the desired speed level.

The design speed must never be lower than the speed limit. It means that the design speed functions as an upper limit of the speed limit. On the other hand, it is not wise to have a speed limit which is much lower than the design speed of a road. This may damage the credibility of the speed limit and therefore does not contribute to highway safety. Moreover, the highway consistency issue is important for determining the posted speed limit along similar sections in the highway network and for governing the design speed on interurban highways.

According to British highway design guidelines (DMRB 1993, NRA 2007) design Speed which is arranged to certain bands (120, 100, 85, 70, 60, 50 km/hour) compared to AASHTO (2011), shall be derived from Figure 1.

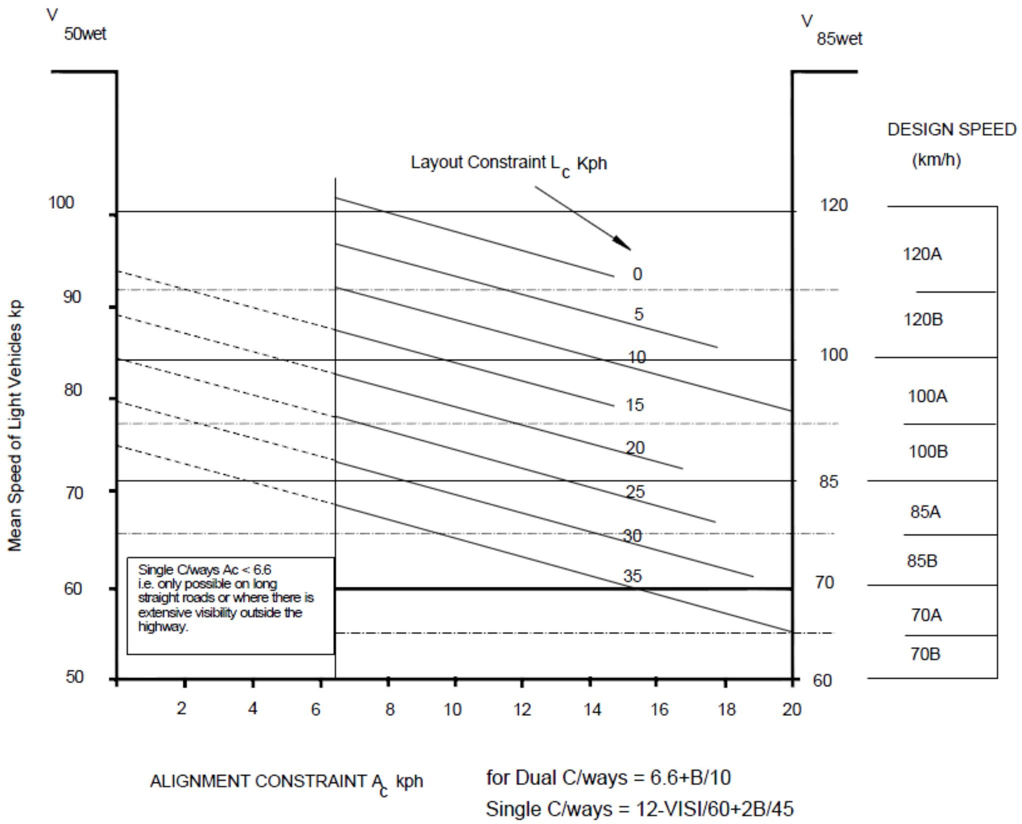

Fig. 1. Selection of design speed in rural roads according to British highway design guidelines (DMRB 1993, NRA 2007)

Figure 1 shows the variation in design speeds for a given alignment constraint (AC) which describes the frequency of curves, layout constraint (LC) which measures the degree of constraint imparted by the road cross section, and frequency of junctions and accesses, and 50th and 85th percentile speeds. The bands of the design speed are arranged with suffixes A and B which indicate the higher and lower categories of each band. For Urban roads the British guidelines connects the mandatory speed limits to the design speed bands. The difference between mandatory speed limit and design speed is ranged between 12 and 4 km/hour (Table 1).
The concept of designing roadways in a manner that will force drivers to operate at a desirable speed (target speed concept) represents a speed management approach where the objective is not to simply reduce speeds but to design the roadway alignment for an appropriate operating speed. This is a concept of "self explaining, self enforcing" road where roads are designed for a specific purpose or function, and therefore, make an acceptable selection of the design speed which could anticipate the operating speed.
Still, the relationship between the design speed and posted speed limit requires further systematic research due to lack of knowledge on the relationship between the design speed and highway safety (Stamatiadis 2005).

Table 1. Correlation between mandatory speed limit and design speed (DMRB 1993)

Speed limit (km/hour)	48	64	80	96
Design speed band (km/hour)	60B	70A	85A	100A

2. Stopping Sight Distance (SSD)

Stopping sight distance (SSD) is the most important of the sight-distance considerations since sufficient SSD is required at any point along the roadway. SSD is the minimum distance required to stop a vehicle traveling at or near the design speed before it reaches an object (vehicle or any other obstruction) in its path.
The stopping sight distance has two components: (1) the distance traveled during the driver's reaction time, usually 2.5 seconds for open roadways; (2) the distance traveled during braking by implementing equivalent deceleration rate (d, meter/sec^2).

$$SSD = \frac{t_R}{3.6} \cdot V_d + \frac{V_d^{\,2}}{2 \cdot 3.6^2 \cdot d} \tag{1}$$

where:
SSD – Minimum stopping sight distance (m)
V_d – Design speed (km/hr).
d – Deceleration of passenger cars (m/s^2), 3.4 m/sec^2.
t_R (or PRT) – Perception reaction time (s), usually 2.5 seconds.

This formula of theoretical stopping sight distance assumes level terrain. Ascending grade decreases the SSD, and descending grade increases the SSD. Trucks, in general, require longer stopping sight distance than passenger cars for a given design speed due to inferior braking characteristics (Bassan 2012). Recent studies (Gargoum et al. 2018, Gavran et al. 2016) have analyzed the difference between available sight distance (ASD), based on road geometry and theoretical sight distance (Eq. 1).

2.1. Perception Reaction Time (PRT)

Perception reaction time (PRT) or brake reaction time (documented in AASHTO) is "the interval from the instant that the driver recognizes the existence of an obstacle on the roadway ahead that necessitates braking until the instant that the driver actually applies the brakes" (AASHTO 2004, 2011). Fambro et al. (1997) and other older additional studies [Johansson & Rumar 1971, Normann 1953] show that PRT of 2.5 seconds for stopping sight situations fits the capabilities of most drivers, including older drivers (or tired to certain extent). This PRT is not adequate for the most complex conditions encountered in actual driving such as ramp terminals of interchanges while driving along a freeway and also prior to multiphase at grad intersections while driving along arterials or rural highways with signalized intersections.
Campbell et al. (2012) explored the variation in PRT under favorable (in general good visibility, where the hazard is visible clearly to the driver) and unfavorable conditions (poor visibility, generally night time or hidden hazards by the surroundings during day time). In good visibility the mean PRT was estimated as 1.6 seconds and in poor visibility – 5.0 seconds.
An absolute minimum value of 2.0 seconds was recommended in the following cases: (1) the reaction

time adapts commuter (familiar) traffic, (2) the design speed is less than or equal to 70 km/hr (low-speed rural areas (Transit 2003). The Canadian design guidelines (TAC 1999) provided three categories for perception-reaction time. The first range of 0.5-2.0 seconds corresponds to alerted drivers to simple stimulus. The second category, 2.5 seconds, is representative of the 90^{th} percentile of driver situations. The third range 3.0-4.5 seconds corresponds to the reaction of unalerted drivers to complex or inconspicuous stimuli. Durth and Bernard (2000) showed that 1.8 seconds exceeds the 95^{th} percentile value. Since this result was based on an experiment in which the drivers were in an "outstanding watchfulness" they recommended on 2.0 second for perception-brake reaction time value. Table 2 presents a comparison of the PRT parameter in several highway geometric design policy guidelines: Australia (Austroads 2009), and PIARC (2003), propose possible lower PRT values than the conventional value of 2.5 seconds in certain circumstances and UK (NRA 2003) proposes a lower value of 2.0 seconds.

Table 2. Typical comparison of perception-reaction time (sec) parameter in open roadways

Country	Open roadways
Australia (Austroads, 2009)	2.5 sec: Absolute minimum for rural highway (high design speed). 2.0 sec: for urban arterial (high design speed) or alerted drivers on rural highways. 1 sec: Constrained condition with maximum vigilance.
Australia (Austroads 2003)	2.5 sec: standard for rural roads. 2.0 sec: minimum reaction time where it may not be practicable to design for a 2.5 second reaction time, such as low-speed alignments in difficult terrain.
Ireland (NRA 2003)	2 sec
USA (AASHTO 2011)	2.5 sec
PIARC (2003), TAC 1999	2.5 sec like Canada (TAC 1999) for 90% of drivers. 0.5-2.0 sec: for alerted and skilled drivers. 3.0-4.5 sec: for non-skilled drivers.
RAA (2008), Germany	2.0 sec: for rural motorways

The standard design value of PRT that is used in rural roads is therefore 2.5 seconds (AASHTO 2004, 2011). This design value includes a perception time of 1.5 seconds (until the driver comes to realization that the brakes must be applies) and additional 1.0 second for reaction until the driver practically applies the brakes. Nonetheless, newer practice from Germany (RAA 2008), UK (NRA 2003), France (PIARC 2003) and UK (NRA 2003) and Australia (Austroads 2009, 2003) might support to reduce this standard PRT design value to 2.0 seconds.

2.2. Deceleration rate (d) and Longitudinal Friction Coefficient (f)

Studies documented by Fambro et al. (1997) have shown that most drivers decelerate at 4.5 m/s^2 (0.459•g) during braking to an unexpected object in the roadway. Experiments showed that 90% of drivers decelerates at rates greater than 3.4 m/s^2, 0.347•g (AASHTO 2004, 2011). The longitudinal friction coefficient in wet pavement surfaces and the modern vehicle braking capabilities enable larger equivalent deceleration rate, than this deceleration rate, e.g. 3.4-4.5 m/sec^2 (Bassan 2012, AASHTO 2011). Also, Durth and Bernhard (2000) recommended that the deceleration threshold for calculating the sight distance would be 4.5 m/s^2 after Considering the antilock braking systems (ABS) and wet pavement surface.

These rely on an improvement of the quality of tires, which strongly affects the skidding longitudinal friction coefficient between a wet pavement and the tires; and the quality of the pavement of most highways. Campbell et al. (2012) investigated the effects of favorable and unfavorable conditions on deceleration rates. Good conditions (in terms of deceleration aspect) were categorized as straight road segments, vehicle tyres in good conditions, passenger car, and dry or wet pavement. Unfavorable conditions were classified as conditions where stopping maneuver occurred in a curve or downgrade and where the pavement conditions were poor. In good pavement conditions the deceleration rate was estimated as 5.4 m/sec^2 and in poor visibility – 4.2 m/sec^2.

Additional recent measurements from Germany (reported in Brilon and Lippold 2005) suggested that the maximum deceleration was not significantly affected by the speed level as long as the tires fulfilled the technical safety requirements. The frictions measured differed systematically for vehicles with and without anti-lock equipment ABS. The new German guidelines, therefore, recommends on deceleration rate of 3.7 m/s^2 for existing roads to be renovated and 4.3 m/s^2 for new roads. The assumption is that operation of new highways will occur after most vehicles are equipped with ABS braking system.

Additional reasons for better pavement quality and therefore improved longitudinal friction coefficients are periodical maintenance activities such as pavement cleaning and washing and pavement stratification and scrubbing control. These activities reduce the skidding component of pavement, presuming that the asphalt concrete mixture is of high quality such as stone mastic asphalt (SMA).

AASHTO (2004, 2011) recommends on a conservative design value 3.4 m/s^2 as a reasonable deceleration rate for obtaining the stopping sight distance and it no longer provides the friction coefficient design values which depend on the design speed. This design value is considered a comfortable deceleration for most drivers (AASHTO 2004, 2011) for stopping but also for stopping maneuver type in implementing decision sight distance (section 3). This design value might be modified by taking into account modern braking systems; the quality of tires; and the quality of the pavement; especially for the lower design speeds (Bassan 2012).

2.3. Additional aspects of stopping sight distance (SSD) from recent literature

Additional implementations for analyzing SSD (and specifically available SSD) based on recent studies include reliability analysis, 2D and 3D methods and GIS applications, and operational (and dynamic) models.

Reliability analysis and HSO relevance

Probabilistic design approaches have been applied in several studies related to highway design and highway safety by using reliability concepts (Navin 1990, 1992). The reliability analysis approach was also implemented in other studies such as Ismail and Sayed (2012) and Ibrahim et al. (2012) in order to select a suitable combination of cross section elements with restricted sight distance to result in reduced collisions and acceptable risk levels.

Hussein and Sayed (2014) applied reliability analysis (stochastic approach) to take into consideration uncertainty associated with geometric design parameters. They calibrated design charts for Horizontal Sightline Offset- HSO, or in other words: the middle ordinate – M, at different probabilities of non-compliance (Pnc) levels.

In highway geometric design the use of Pnc characterizes the probability that the design does not meet the standard design requirement. Hussein and Sayed used a limit state function which is the difference between ASD (available stopping sight distance) and the demanded stopping sight distance (SSD). The Pnc was generated from this function. SSD is presented in Eq. 1. ASD is the accurate horizontal sight distance formula assuming SSD is shorter than the horizontal curve length (L). The preferred target Pnc is the road designer's choice based on the design policy.

The calibration of horizontal radius and HSO for different Pnc's showed that the current design guides are conservative especially at sharp radii and high design speed (Hussein and Sayed 2014). It appears that these results are somehow subjective because increasing the risk (i.e. increasing the target Pnc) can be performed by deterministic modifications such as reducing the perception reaction time (PRT) or increasing the friction coefficient (Eq. 1) based on the prevailing traffic and pavement conditions and specific driving behavior characteristics. SSD and HSO, when approaching a horizontal curve and within the horizontal curve were studied by Wood and Donnell (2014). These scenarios were investigated in different combinations of speed limit, curve radius, and superelevation. Their results showed that the probability of non-compliance (i.e. drivers would not have adequate sight distance to see react and stop before reaching an object in the roadway) when approaching the horizontal curve is greater than within the curve.

3D models and GIS applications

Sarhan and Hassan (2012) analyzed available stopping sight distance (SSD) on horizontal curves with roadside or median barriers in 3 Dimension (3D) combined alignments by using finite element technique software. They found that the available SSD increases as the overlapping crest vertical curve becomes flatter or as the overlapping sag vertical curve becomes sharper.

Additional studies aimed to evaluate and optimize the actual sight distance in real driving conditions by

3D models. Nehate and Rys (2006) examined the intersection of sightline with the elements representing the road surface by an algorithm which combines horizontal and vertical alignment and automatically calculate sight-distance profile along any given highway for which Global Positioning System's (GPS's) data are available. Results of actual data have shown the ability to identify sight-distance restrictions. Similarly, Kim and Lovell (2010) used computational geometry and thin plate Spline interpolation to represent the road surface and eventually determine the maximum available sight distance.
Castro et al. (2011) implemented a procedure supported by Geographic Information Systems (GIS) in order to determine highway distances visible to the driver in a two-lane highway section. The GIS advantage in highway sight distance analysis is using data sources that besides the terrain include obstacles like trees or buildings that could reduce the driver sight distance. This procedure can perform highway sight distance analysis of existing highways where highway design information (tangents, horizontal and vertical curves) is not available or is not the latest (e.g. in many already built highways). Castro et al. (2011) showed that sight distances obtained by GIS approach and the classic highway design approach are statistically the same.
Jha et al. (2011) proposed a 3-D design methodology which is capable of "efficiently measuring the sight distance for different superelevation, day and nighttime conditions, and obstructions".
Moreno et al. (2014) maximized the available stopping sight distance (SSD) at crest vertical curves overlapped with horizontal curves in two lane highways. They used a finite element method algorithm to generate an SSD profile. The available SSD profile indicated the available SSD of each evaluated position of the driver. The resulted available SSD could finalize certain alignment zones where vertical and horizontal curve could be improved if the minimum available SSD was lower than required among these curves. This study concluded that the vertical curve curvature in meters ($K=L/A=R_v/100$, i.e. the horizontal distance needed to make a 1 percent change in gradient), and the available horizontal radius (R_h) affect the available SSD. The optimal proportion of Kv/Rh which maximizes the available SSD is generally ranged between 0.05 to 0.15. The study also revealed a negligible impact of the offset between horizontal and vertical curves on available SSD. Additionally, the layout visibility got restricted to the point where the superelevation changed its sign (Moreno et al. 2014).
Mavomartis et al.(2015, 2012) proposed an "accurate SSD control method" which relates the 3D alignment of the roadway (horizontal and vertical) to the dynamics of vehicle path along the roadway based on the difference between the available SSD and the "demanded" SSD. The method was applied to left turn horizontal curve divided highway. This research finally suggested that by increasing the object height (i.e. vehicle tail lights) to 1.08 meters, the design consistency, and driver expectations can be satisfied by avoiding non-uniformed posted speed limits and uneconomical lateral road widening. Ismail and Sayed (2007) presented a finite element approach of a 3D algorithm that calculates ASD for integrated vertical and horizontal alignments. This algorithm parameterizes horizontal elements such as pavement edge, median edge, and side slope relative to the centerline.

Operational and dynamic and geometric design aspects

Gavran et al. (2016) concentrated on calculating the required sight distance (RSD) and ASD. RSD calculation is based on operating speed analysis and ASD determination is based on illustrative 3D techniques (triangulated roadway models: 3D digital elevation models (DEM) methods). Unlike the design speed which is a constant value, the operating speed changes along the road (reaching higher levels in greater radii and vice versa). Therefore SSD which is calculated for a certain design speed (Eq. 1) is not sufficient for a vehicle travelling at a higher speed (operating speed) to perform a sudden stop in an emergency (Gavran et al. 2016). It is important to consider realistic operating speed levels at a design early stage to provide the RSD. However, Gavran's methodology does not refer to balanced geometric design or design consistency.
Gaca and Pogodzińska (2017) emphasized the legitimacy of speed management and the use of local speed limits in areas of increased accident risk. The influence of speed management measure on road safety can be indirectly achieved through estimating road safety measures related to traffic crashes and crash risk (Crash Modification factors) based on accident location (local, regional and national roads) and accident types and their severity. Estimating the

impact of any speed management measure on road safety can also be made by using an intermediate criteria, such as change of vehicle speed caused by a particular safety measure (Gaca and Pogodzińska 2017). AASHTO (2011), however, states that the design speeds are supposed to be logical to anticipate the operating speed and be practical enough to attain safety mobility and efficiency.

Xia et al. (2016) analyzed drivers braking process and inferred cornering braking distance based on vehicle kinematics and simulation software in order to resolve the minimum value of horizontal curve SSD. They concluded that the vehicle needs longer curve SSD in order to meet the requirements "cornering braking stability". Also Cheng et al. (2011) revised the SSD calculation by finding the critical conditions of vehicles driving at curves. Obviously, driving at curve section with superlevation is likely accident prone area compared to tangent section. Similarly to Xia et al.(2016), Cheng et al. (2011) concluded that driving at curves requires longer SSDs than those at straights.

2.4. Changes in parameters used for SSD calculation based on AASHTO (1940-2011)

The basic SSD calculation model presented in Eq. 1 was eventually formulized by AASHTO first version in 1940. Adjustments in the model parameters have been made over the past decades as presented in Table 3.

Table 4 presents computed values of stopping sight distance for the design speed range: 30-140 km/hour based on Eq. 1 and AASHTO (2011) recommended PRT and deceleration rate (d) parameters.

Table 3. Changes in SSD parameters based on AASHTO previous versions (partially based on Fambro et al. (1997), Cheng et al. (2011)

AASTO reference (year, title)	**Speed considered**	**Perception Reaction Time, PRT (sec)**	**Design pavement/ stop**	**Friction factors (d/g) or deceleration rate, d, (m/sec^2)**	**Driver eye height (m)**	**Object height (m)**
1940, a policy on sight distance for highways.	Design speed.	3.0 at 48.3 km/hour; 2.0 at 112.6 km/hour.	Dry pavement/ locked wheel stop	Ranges from 0.50 at 48.3 km/hour, to 0.40 at 112.6 km/hour.	1.37	0.10
1954, A policy on geometric design of rural highways.	85-95% of design speed.	2.5	Wet pavement/ locked wheel stop	Ranges from 0.36 at 48.3 km/hour, to 0.29 at 112.6 km/hour.	1.37	0.10
1965, A policy on geometric design of rural highways.	80-93% of design speed.	2.5	Wet pavement/ locked wheel stop	Ranges from 0.36 at 48.3 km/hour, to 0.27 at 112.6 km/hour.	1.14	0.15
1971, A policy on geometric design of highways and streets.	Minimum: 80-93% of design speed. Desired: design speed	2.5	Wet pavement/ locked wheel stop	Ranges from 0.35 at 48.3 km/hour, to 0.27 at 112.6 km/hour.	1.14	0.15
1984, 1990, A policy on geometric design of highways and streets.	Minimum: 80-93% of design speed. Desired: design speed	2.5	Wet pavement/ locked wheel stop	Slightly higher at higher speeds than 1970 values.	1.08	0.15
1994, A policy on geometric design of highways and streets.	Minimum: 82-100% of design speed. Desired: design speed	2.5	Wet pavement/ locked wheel stop	Ranges from 0.40 at 30 km/hour to 0.28 at 128 km/hour.	1.08	0.15
2001, 2004, 2011, A policy on geometric design of highways and streets.	Design speed	2.5	Wet pavement/ locked wheel stop	**Deceleration rate: 3.4 m/sec^2** (friction factor: 0.3466).	1.08	**0.60**

Table 4. SSD computed values based on design speed and AASHTO (2011) parameters

Design speed (km/hour)	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**	**130**	**140**
SSD computed, based on AASHTO (m) *	32	46	64	83	105	129	153	183	214	247	283	320

AASHTO parameters: PRT=2.5 seconds, d=3.4 m/sec²

3. Decision Sight Distance (DSD)

Stopping sight distances are usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under normal circumstances. However, greater distances may be needed where drivers must make complex or immediate decisions, where information is difficult to perceive, or when unexpected or unusual maneuvers are needed (AASHTO 2004, 2011). These unusual maneuvers may cause the driver to vary with the operating speed rather than to stop, and thus, the outcome is a longer sight distance than stopping sight distance. This distance provides the necessary time for drivers to anticipate changes in highway design features (such as intersections, interchanges, lane drops, etc.) or a potential hazard in the roadway, and therefore, perform an essential maneuver.

Decision sight distance (DSD) is defined as the distance at which drivers can detect a hazard or signal in a cluttered roadway environment, recognize it (or its threat potential), select the appropriate speed and path, and perform the required action safely and efficiently (Alexander & Lunenfeld 1975).

The decision sight distance (DSD) enables a maneuver which is less risky than the braking maneuver of stopping sight distance until a complete stop. When the driving situation causes the driver to detect an unexpected or difficult to perceive or incorrectly perceived information source, DSD should be applied.

Examples where driving situation requires decision sight distance are:

- Complex interchanges or intersections (prior to merging and diverging ramp terminals).
- Changes in cross-section (e.g. toll plaza and lane drop or lane addition prior to a signalized intersection).
- Locations and situations where unusual or unexpected maneuver occur (e.g. weaving zones).
- Areas which require multiple decision making capabilities from the driver almost instantaneously (e.g. road elements and abrupt changes in the alignment profile, traffic control devices, warning or guidance areas, advertising, sudden increase of traffic, traffic queue).
- Construction zones.
- Alleviating difficulties created by trucks and heavy vehicles entering the traffic stream.
- Maneuvering lane change due to bicycle lanes or bicycle paths especially prior to intersections.

McGee (1979) developed guidelines on DSD values based on "hazard avoidance model". McGee (1979) recommended a range of decision sight distances values as a function of the design speed. These values are based on the summation of the time differences: detection and recognition tinme , decision and response initiation time, and maneuver time, as presented in Table 5.

Table 5. Hazard avoidance model – DSD values, m (McGee 1979)

Design speed (km/hr)	Time intervals (seconds)				Decision sight distance (m)	
	Pre maneuver		Maneuver	Total time	computed	rounded
	Detection and recognition	Decision and response initiation				
40	1.5-3.0	4.2-6.5	4.5	10.2-14.0	113-156	120-160
60	1.5-3.0	4.2-6.5	4.5	10.2-14.0	170-233	170-230
80	1.5-3.0	4.2-6.5	4.5	10.2-14.0	227-311	230-310
100	2.0-3.0	4.7-7.0	4.3	11.2-14.5	306-397	310-400
120	2.0-3.0	4.7-7.0	4.0	10.7-14.0	357-467	360-470
140	3.0-3.0	4.7-7.0	4.0	10.7-14.0	416-544	420-540

The maneuver time is assumed to be lane change such that the time of speed reduction was neglected. Bassan (2011) proposed a new DSD model which consists of three driving maneuvers stages:
(1) The pre-maneuver stage.
(2) The braking action from the free flow speed (the design speed for the purpose of highway design) to the maneuver speed.
The maneuver operation.

$$DSD = \frac{5.5}{3.6} \cdot V_D + \frac{V_D{}^2 - V_M{}^2}{2 \cdot 3.6^2 \cdot d} + \frac{T_M}{3.6} \cdot V_M = \\ = 1.53 V_D + \frac{V_D{}^2 - V_M{}^2}{25.92 \cdot d} + \frac{T_M}{3.6} \cdot V_M \quad (2)$$

where:
DSD – decision sight distance (m),
V_D – design speed (km/hour),
d – average deceleration rate (m/sec^2): 4.3-3.7 for design speed 60-120 km/hour,
T_M – maneuver time (sec): 4.5-3.5 for design speed 40-130 km/hour,
V_M – average maneuver speed: lower than the design speed by 10-60 km/hour.

3.1. AASHTO (2011) DSD model

AASHTO (2004, 2011) proposes two basic avoidance types for decision sight distance:
Type I: Comfortable stopping maneuver, which requires pre-maneuver time (Eq. 3), which is higher than the PRT parameter (2.5 seconds) implemented for stopping sight distance.
Type II: Changing in speed, path, or direction, which requires a pre maneuver time and maneuver time (t_M) without a braking component (Eq. 4). The tradeoff of not including a braking component is the inclusion of an increased fixed value of pre maneuver plus maneuver time (t_M). The maneuver time component of t_M ranges between 3.5-4.5 seconds (based on McGee (1979)). The upper limit corresponds to the lowest bunch of design speeds (30, 40, 50 km/hour) and the lower limit is appropriate with the highest bunch of design speeds (130, 140 km/hour). This concept complies with AASHTO (2004, 2011).

$$DSD(I) = \frac{PMT}{3.6} \cdot V_d + \frac{V_d{}^2}{2 \cdot 3.6^2 \cdot d} \quad (3)$$

$$DSD(II) = \frac{t_M}{3.6} \cdot V_d \quad (4)$$

where:
DSD(I) – decision sight distance, Type I (m).
DSD(II) – decision sight distance, Type II (m),
V_d – design speed (km/hr),
d – deceleration of passenger cars (m/s^2), 3.4 m/sec^2 (identical to stopping sight distance, Eq. 1),
PMT – pre-maneuver time (seconds), for DSD(I),
t_M – total pre maneuver time + maneuver time (seconds), for DSD (II).

Each maneuver type corresponds to three highway categories: urban, suburban, and rural. For maneuver type I: avoidance maneuver A is used for rural road, avoidance maneuver A1 is used for suburban road, and avoidance maneuver B is used for urban road.
For maneuver type II: avoidance maneuver C is used for rural road, avoidance maneuver D is used for suburban road, and avoidance maneuver E is used for urban road.
Shorter DSD lengths would generally be used for rural roads and suitable with higher design speeds and for type I avoidance maneuver types (stopping). The DSD parameters (PMT and t_M) are lower for potentially less complex rural conditions than for urban driving situations
Urban road is implemented in this study for a maximum design speed of 90 km/hour and not 130 km/hour which is proposed by AASHTO (2004, 2011).
Pre-maneuver time (PMT) for DSD(I):
The pre maneuver time depends on the highway category:
Avoidance type A: PMT=3.0 sec (rural highway).
Avoidance type A1: PMT=6.0 sec (suburban highway). This avoidance type is not included in AASHTO (2004, 2011).
Avoidance type B: PMT=9.1 sec (urban road).
The pre-maneuver times do not depend on design speeds.
Total pre maneuver time plus maneuver time (t_M) for DSD(II):
The total pre maneuver time plus maneuver time (t_M) depends on the highway category similarly to DSD(I):

Avoidance type C:
10.2≤ t_M ≤11.2 seconds (rural highway). The highest threshold (11.2 seconds) corresponds to design speed, Vd=50 km/hour. The lowest threshold (10.2 seconds) corresponds to design speed, Vd=130 km/hour.
Avoidance type D:
12.1≤ t_M ≤12.9 seconds (suburban highway). The highest threshold (12.9 seconds) corresponds to design speed, Vd=50 km/hour. The lowest threshold (12.1 seconds) corresponds to design speed, Vd=130 km/hour.
Avoidance type E:
14.0≤ t_M ≤14.5 seconds (suburban highway). The highest threshold (14.5 seconds) corresponds to design speed, Vd=50 km/hour. The lowest threshold (14.0 seconds) corresponds to design speed, Vd=90 km/hour.
Table 6 presents the pre maneuver time (PMT) and total pre maneuver plus maneuver time (t_M) for all DSD avoidance types reviewed in this section: A,A1,B,C,D,E.
Table 7 summarizes DSD computed values of the avoidance types reviewed in this section: A,A1,B,C,D,E based on the maneuver times parameters presented in Table 4. The DSD values are not totally equal to the values presented in AASHTO (2011), due to certain differences in the maneuver time parameters as a function of the design speed.

3.2. DSD-SSD patterns by AASHTO (2004, 2011) models

The assessed SSD and DSD computed values (Table 4, Table 7) function as an input for calibrating a model which directly (empirically) formulates the relationship between DSD and SSD for every DSD avoidance maneuver type.
Figure 2 presents a scatter plot of the decision sight distance (y axis) and the stopping sight distance (x axis) for the six avoidance types presented (Type I: A,A1,B, and Type II: C,D,E). These scatter plots were constructed by employing data points for design speeds within intervals of 2 km/hour and interpolation of SSD's and DSD's parameters accordingly.

4. DSD-SSD Empirical Modeling Based on AASHTO Avoidance Maneuver Types

A model that reproduces the relationship between DSD and SSD has not been found in the literature except a simplified approximation documented in the British highway design guidelines [DMRB 1993, NRA 2007]. These guidelines propose that the distance required for the driver to reach a decision point is 1.5 multiplied by the "desirable minimum stopping sight distance". This decision point could be located upstream of:
(1) a stop line or yield line along the major road until the intersection with a minor road (intersections).
(2) a stop line or yield line along the major road until a roundabout (roundabouts).
(3) the start of the diverge taper to the back of the diverge nose (diverge ramp terminal).
(4) the back of the merge nose to the end of the merge taper (merge ramp terminal).

Table 6. A summary of DSD maneuver time parameters (PMT, t_M) for DSD avoidance types A,A1,B,C,D,E, depending on the design speed:

Vd (km/hour)	**PMT: type A (rural) (sec)**	**PMT: type A1 (suburban), (sec)**	**PMT: type B (urban), (sec)**	**t_M: Type C (rural), (sec) ***	**t_M: Type D (suburban), (sec) ***	**t_M: Type E (urban), (sec) ***
30	3	6	9.1	11.2	12.9	14.5
40	3	6	9.1	11.2	12.9	14.5
50	3	6	9.1	11.2	12.9	14.5
60	3	6	9.1	11.075	12.8	14.375
70	3	6	9.1	10.95	12.7	14.25
80	3	6	9.1	10.825	12.6	14.125
90	3	6	9.1	10.7	12.5	14.0
100	3	6	-	10.575	12.4	-
110	3	6	-	10.45	12.3	-
120	3	6	-	10.325	12.2	-
130	3	6	-	10.2	12.1	-
140	3	6	-	10.2	12.1	-

***** *t_M includes a maneuver time component of 3.5-4.5 seconds (based on McGee 1979)*

Table 7. A summary of DSD computed values of avoidance types A,A1,B,C,D,E, depending on the design speed

Vd (km/hour)	DSD(I): type A (rural) (m)	DSD(I): type A1 (suburban), (m)	DSD(I): type B (urban), (m)	DSD(II): Type C (rural), (m)	DSD(II): Type D (suburban), (m)	DSD(II): Type E (urban), (m
30	36	61	87	94	108	121
40	32	85	120	125	144	162
50	71	112	155	156	180	202
60	91	141	193	185	214	240
70	114	173	233	213	247	278
80	140	206	275	241	280	314
90	167	242	320	268	313	350
100	197	281	-	294	348	-
110	229	321	-	320	376	-
120	264	364	-	345	407	-
130	301	409	-	369	437	-
140	340	456	-	397	471	-

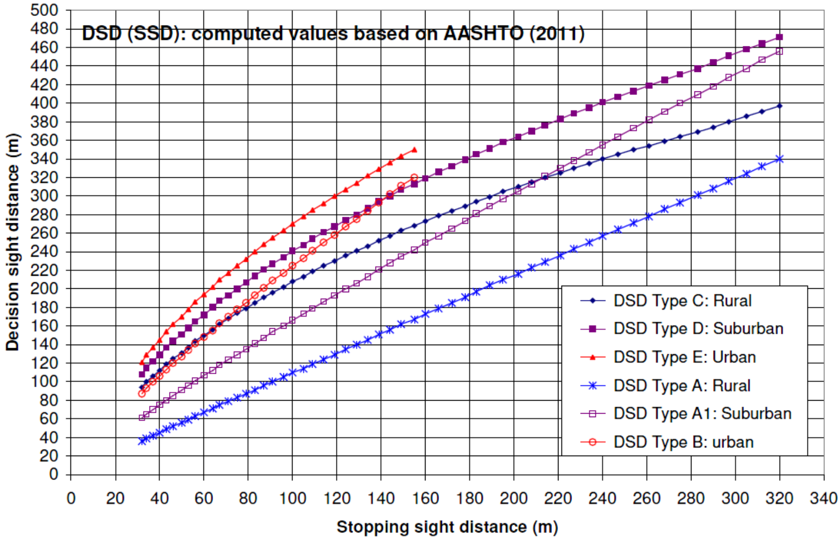

Fig. 2. A typical scatter plot of DSD Vs. SSD for avoidance types: A, A1, B (Type I); C, D, E (Type II) based on AASHTO 2011

The British simplified approximation of the relationship between DSD and SSD is uniform without a categorization of avoidance maneuver types and highway classification. Both British and U.S. (AASHTO) SSD models are based on the design speed. The British SSD design values are based on narrow bands of design speeds compared to AASHTO, even though these bands could be adjusted to smaller design speed increments (e.g. 10 km/hour) by a linear approximation.

By examining the scatter-plots presented in Figure 2 it appears that a natural logarithmic model would fit the data points of the six DSD types presented, practically well. The proposed general form of a model that reflects the relationship between DSD and SSD is as follows:

$$\ln(DSD) = a + b \cdot \ln(SSD) \quad (5)$$

or:

$$DSD = e^{a+b\cdot\ln(SSD)} \quad (6)$$

The general form of the model presented in Eq. 4 was calibrated by regression analysis according to the data points presented on Figure 2. The resulted calibrated parameters (a, b) of six calibrated models for the six DSD types, A, A1, B, C, D, E, correspondingly is presented in Table 8. Also presented in Table 4 are the resulted coefficients of determination (R^2) of these models.
The resulted coefficients of determination (R^2) are almost equal to 1 for all calibrated models. This means that almost all the variation in the dependent variable (DSD) is explained by the regression line. The reason is that the data points themselves are based on specific modeling of SSD and DSD based on physical and driver behavior parameters such as: (1) design speed, perception reaction time, and equivalent deceleration rate: for SSD; and (2) design speed, equivalent deceleration rate, pre-maneuver time, maneuver time,: for DSD. Nonetheless, the exact form of the models presented in Eq. 5 produces a simple form for the relationship between DSD and SSD for the six DSD types, which is the major purpose of this study. Figures 3-8 presents graphically the resulted relationship between DSD and SSD for DSD avoidance maneuver types A, A1, B, C, D, E. The figures include also a line that shows the simplified ratio (1.5) between DSD and SSD based on the British road design guidelines [DMRB 1993, NRA 2007].

Table 8. Parameters' summary of the calibrated models, SSD vs. DSD, for six avoidance maneuver types based on AASHTO (2011) DSD models

Parameter / **Avoidance Maneuver Type**	**a**	**b**	**R-Squared**
Type I: A	0.235812	0.96892653	0.9999622
Type I: A1	1.11484503	0.867976622	0.9999803
Type I: B	1.655151402	0.816129034	0.9998994
Type II: C	2.524850747	0.604686581	0.996084
Type II: D	2.602365315	0.620465429	0.9970845
Type II: E	2.553115245	0.659742958	0.9978024

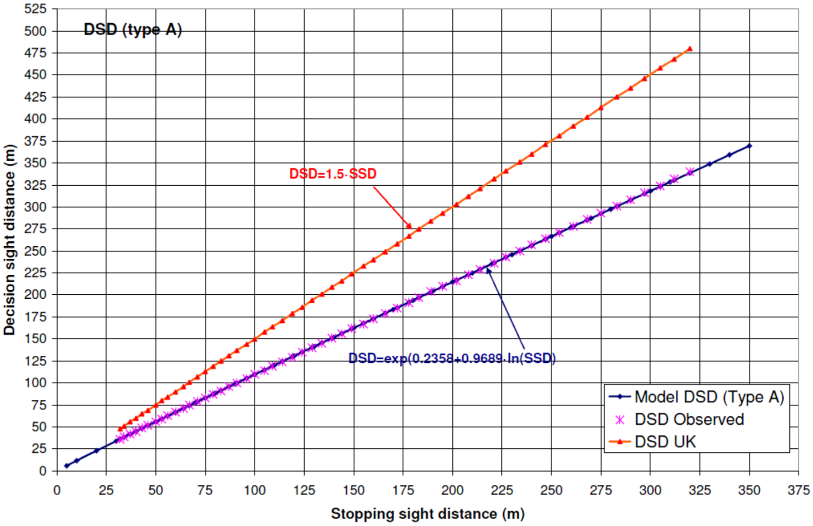

Fig. 3. Graphical presentation of the relationship between DSD and SSD for AASHTO maneuver type A: comfortable stopping, rural road (proposed model and British guidelines)

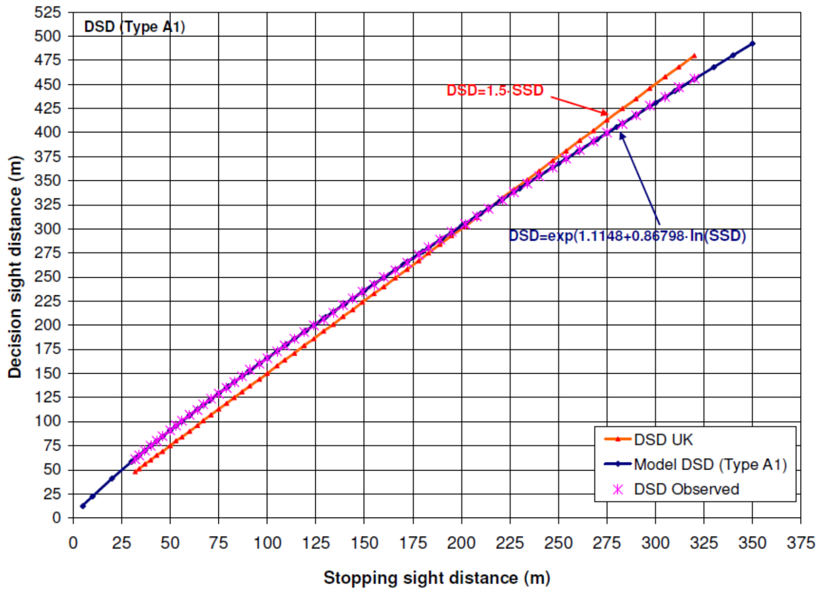

Fig. 4. Graphical presentation of the relationship between DSD and SSD, type A1: comfortable stopping, suburban road (proposed model and British guidelines)

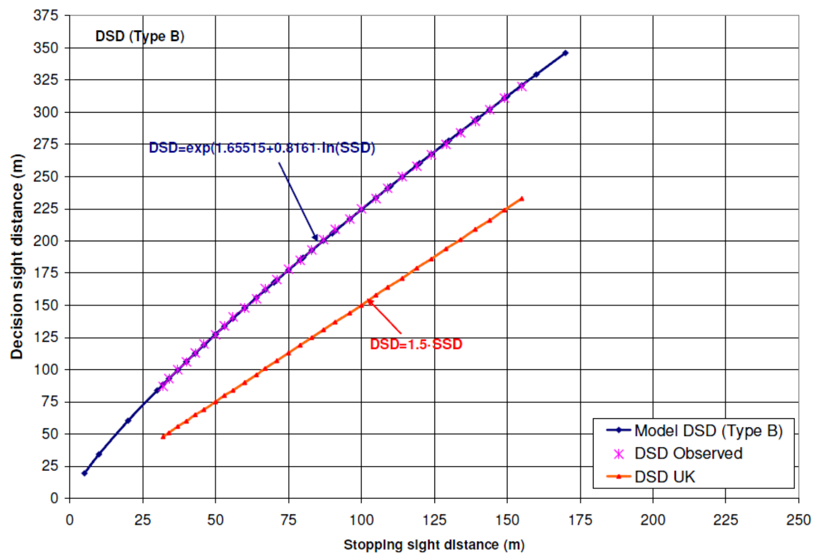

Fig. 5. Graphical presentation of the relationship between DSD and SSD for AASHTO maneuver type B: comfortable stopping, urban road (proposed model and British guidelines)

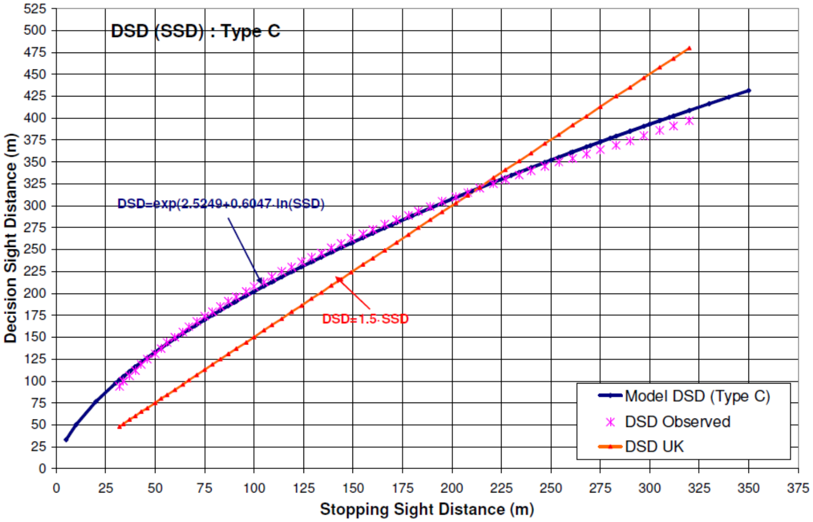

Fig. 6. Graphical presentation of the relationship between DSD and SSD for AASHTO maneuver type C: speed/path/direction change, rural road (proposed model and British guidelines)

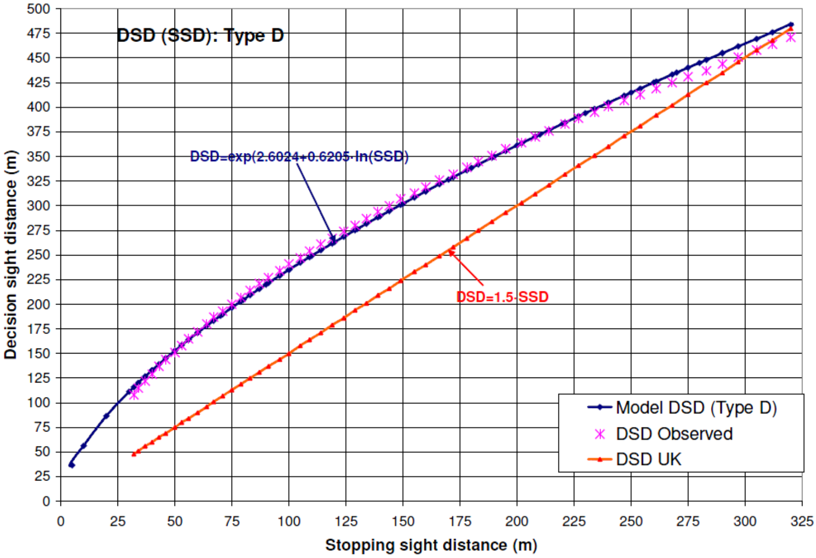

Fig. 7. Graphical presentation of the relationship between DSD and SSD for AASHTO maneuver type D: speed/path/direction change, suburban road (proposed model and British guidelines)

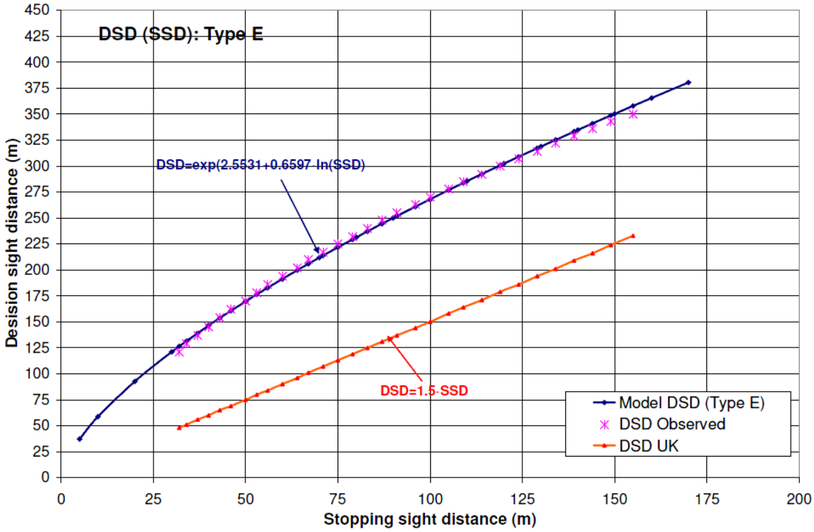

Fig. 8. Graphical presentation of the relationship between DSD and SSD type E: speed/path/direction change, urban road (proposed model and British guidelines)

The graphical plots presented in Figures 3-8 show that only avoidance type A1 (stopping on suburban road) which is not formally included in AASHTO (2004, 2011) is roughly comparable to the simplified approximation proposed by the British highway design guidelines.

5. Summary and further design insights

The paper overviews stopping sight distance and decision sight distance parameters and models which are mostly documented in AASHTO (2011). The DSD which enables a maneuver which is less risky than the braking maneuver of SSD, needs to be implemented before critical points of the road alignment such as un-signalized intersections, interchanges (merging and diverging ramps), acceleration and deceleration lanes, weaving zones, toll plazas, lane drops, abrupt changes in the alignment profile, and warning or guidance areas. Because of additional maneuvering distance needed for safety it is recommended that DSD might be provided prior to critical locations presented or alternatively moving the critical decision points to where satisfactory distances are available.

Six avoidance types of decision sight distance are reviewed. The first three refer to comfortable stopping maneuver (category I) and the other three refer to changing in speed, path, or direction which requires a pre-maneuver time and maneuver time. Each category corresponds to three highway classes: rural, suburban, and urban roads (A, A1, B for category I; and C, D, E for category II respectively. The computed sight distance values are utilized as an input for the major purpose of this study which is calibrating empirical models that reveal the relationship between DSD and SSD. All models were calibrated by a general natural logarithmic formulation. The advantage of this empirical relationship between DSD and SSD is that after computing SSD we can obtain the DSD length of certain avoidance maneuver type and highway class (e.g. rural, urban, suburban) without the need of estimating its relevant maneuver time parameters. Figure 9 introduces a graphical summary of the DSD-SSD empirical models presented in this study. An optional modification for AASHTO "comfortable stopping maneuver" (category I) could include a pre-maneuver time which is not larger than the PRT parameter of SSD (2.5 seconds) but a braking component which has a lower equivalent deceleration rate (lower than 3.4 m/sec^2). Such component could portray a more comfort braking e.g. 0.25g-0.26g , as proposed by Austroads (2009) and UK (NRA 2003).

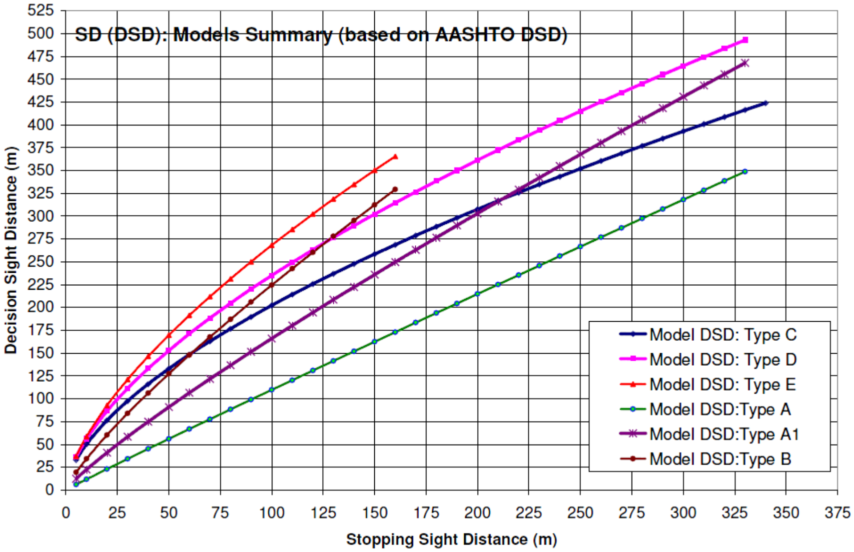

Fig. 9. Graphical presentation summary of DSD-SSD natural logarithmic models for DSD avoidance maneuver types: A, A1, B, C, D, E based on AASHTO

Horizontal and vertical curves may challenge provision of DSD length and also may trigger relocation of non-practical decision zones. Therefore, the highway engineer might consider in such design scenarios the application of traffic control and Intelligent Transportation Systems. The DSD length could be utilized to examine the necessity of advance warning message sign. The sign might assist the driver to reduce the pre-maneuver time component of the DSD specifically the detection and recognition, and reduce the probability of colliding the hazard. If the sighting distance is too short, then a warning sign could inform the driver which maneuver or maneuver alternatives should be considered in order to escape from the obstruction. Nonetheless, supplementing warning signs without sufficient DSD, might not guarantee a reduction in traffic crashes due to restricted DSD (Kostyniuk and Cleveland 1986).

Additional ITS- safety advanced systems could be installed in the vehicle for the driver (Kapusta and Kalašová 2016):

– Collision warning systems, which monitor the roadway ahead, and are supposed to warn a driver when potential danger, such as another vehicle or object, is detected in the same lane.
– Lane departure warning systems, which monitor the position of a vehicle within a lane and are set to warn the driver if the vehicle deviates or is about to deviate outside the lane unexpectedly.
– Rear object detection systems, which detect moving and stationary objects located within a specific area behind the vehicle. Currently used systems can be integrated with other sensors.
– Integration of collision warning system with a current adaptive cruise control system can automatically maintain a minimum interval in relation to the vehicle in front in the same lane. If there is no vehicle ahead, it works as a conventional cruise control so the speed is set by the driver.

Overall, a number of measures might be considered to alleviate DSD restrictions:

increase radii of horizontal curves and vertical curves, increase shoulder width (and therefore HSO) in proximity of horizontal curves, remove sightline obstruction, reduce posted speed limit, provide guidance geometric feature and devices e.g. freeway exit signs, hazard markers, and delineators, and in-vehicle safety and traffic control technological advancements for the driver.

References

[1] AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO), 2004. *A Policy on Geometric Design of Highways and Streets,* 5th Edition. Washington D.C.

[2] AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO), 2011. *A Policy on Geometric Design of Highways and Streets,* 6th Edition. Washington D.C.

[3] ALEXANDER, G. J., LUNENFELD, H., 1975. *Positive guidance in traffic control*. Federal Highway Administration.

[4] AUSTROADS, 2009. *Guide to Road Design, Part 3: Geometric Design*, AGRD03/09, Austroads, Sydney, NSW.

[5] AUSTROADS, 2003. Rural Road Design. *A Guide to the Geometric Design of Rural Roads*. ISBN 0-85588-606-4.

[6] BASSAN, S., 2016. Highway Design Policy Insights for Target Speed: an Israel Perspective. *Traffic Engineering and Control*, 57(4), 155-158.

[7] BASSAN, S., 2012. Review and evaluation of stopping sight distance design - cars vs. trucks. *Advances in Transportation Studies. Special Issue*, 2012, 5-16.

[8] BASSAN, S., 2011. Decision Sight Distance Review and Evaluation. *Traffic Engineering and Control*, 52(1), 23-26.

[9] BRILON, W., LIPPOLD, C., 2005. *A new concept for highway design guidelines in Germany. 3rd international symposium on highway geometric design.*

[10] CAMPBELL, J.L., LICHTY, M.G., BROWN, J. L., RICHARD, C.M., CHRISTIAN, M., GRAVING, J.S., GRAHAM, J., O'LAUGHLIN, M., TORBIC, D., HARWOOD, D., 2012. *Human Factors Guidelines for Road Systems*. NCHRP 600. 2nd Edition TRB.

[11] CASTRO, M., IGLESIAS, L., SANCHEZ, J. A., AMBROSIO, L., 2011. Sight distance analysis of highways using GIS tools. *Transportation Research Part C*, 19(6), 997-1005.

[12] CHENG, J., YUAN, H., SHI, G., HUANG, X., 2011. Revision of calculation of stopping sight distance. *The Baltic Journal of Road and Bridge Engineering*, 6(2), 96–1.

[13] DESIGN MANUAL FOR ROADS AND BRIDGES (DMRB) 1993. *Highway Link Design*, Volume 6 Section 1 Part 1A, NRA TD 9/93, UK.

[14] DURTH, W., BERNHARD M., 2000. Revised Design Parameters for Stopping Sight Distance. *Second International Symposium on Highway Geometric Design*, 410-421.

[15] FAMBRO, B., FITZPATRICK, K., KOPPA, R.J., 1997. *Determination of Stopping Sight Distance*. National Cooperative Highway Research Program (NCHRP), Report 400, Transportation Research Board, Washington D.C.

[16] FITZPATRICK, K., MIAOU, S.P., BREWER, M., 2005, Exploration of the Relationship between Operating Speed and Roadway Features on Tangent Sections, *Journal of Transportation Engineering*, 131(4), 261-269.

[17] GACA, S., POGODZIŃSKA, S., 2017.Speed management as a measure to improve road safety on polish regional roads. *Archives of transport*, 43 (3), 29-42.

[18] GARGOUM, S.A., TAWFEEK, M.H., EL-BASYOUNY, K., KOCH, J.C., 2018. Available sight distance on existing highways: Meeting stopping sight distance requirements of an aging population. *Accident Analysis and Prevention*, 112(2018), 56–68.

[19] GAVRAN, D., FRIC, S., ILIĆ, V., TRPČEVSKI, F., 2016. Sight distance analyses in road design process: Serbian practice. *Transport*, 31(2), 250–259.

[20] HUSSEIN, M., SAYED, T., ISMAIL, K., VAN ESPEN, A., 2014. Calibrating road design guides using risk-based reliability analysis. *Journal of Transportation Engineering*, ASCE, 140(9), 1-6.

[21] IBRAHIM, S.E., SAYED, T., ISMAIL, K., 2012. Methodology for safety optimization of highway cross-sections for horizontal curves with restricted sight distance. *Accident Analysis and Prevention*, 49, 476–485.

[22] ISMAIL, K., SAYED, T., 2012. Risk-optimal highway design: methodology and case studies. *Safety science*, 50(7), 1513–1521.

[23] ISMAIL, K., SAYED, T., 2007. New algorithm for calculating 3D available sight distance. *Journal of Transportation Engineering*, ASCE, 133 (10), 572-581.

[24] JHA, M., KARRI, G., KUHN, W., 2011. New three-dimensional highway design methodology for sight distance measurement. *Transportation Research Record*, 2262, 74-82.
[25] JOHANSSON, G., RUMAR, K., 1971. Drivers' Brake Reaction Times. *Human Factors*, 13(1), 23–27.
[26] KAPUSTA, J., KALAŠOVÁ, A., 2016. A comparison of truck driver safety between the EU and the USA. *Scientific Journal of Silesian University of Technology. Series Transport*, 93, 49-58.
[27] KIM, D.G., LOVELL, D.J., 2010. A procedure for 3-D sight distance evaluation using thin plate splines. *4th International Symposium of Highway Geometric Design*. Valencia. Spain. 6.2010.
[28] KOSTYNIUK, L. P, CLEVELAND, D., 1986. Sight distance, signing, and safety on vertical curves. *ITE journal*, 56(5), 25-28.
[29] LEISCH, J.E. AND LEISCH, J.P., 1977. New Concepts in Design Speed Application. *Transport Research Record*, 631, 4-14.
[30] MAVROMATIS, S., PSARIANOS, B., MERTZANIS, F., VARDAKI, S., 2015. Three dimensional stopping sight distance control on left turn curves of freeways overlapped with crest vertical curves. *5th International Symposium of Highway Geometric Design*. Vancouver. Canada. 6.2015.
[31] MAVROMATIS, S., PALASKAS, S., PSARIANOS, B. 2012. Continuous three-dimensional stopping sight distance control on crest vertical curves. *Advances in Transportation Studies*, 29, 17-30.
[32] NATIONAL ROADS AUTHORITY (NRA), 2007. *Road Link Design*, Volume 6 Section 1 Part 1A, NRA TD 9-07, Ireland.
[33] NATIONAL ROADS AUTHORITY (NRA), 2003. *Design Manual for Roads and Bridges, Guidance on Road Link Design*, Volume 6 Section 1 Part 1A, NRA TA 43-03, Ireland.
[34] MCGEE, H.W., 1979. Decision sight distance for highway design and traffic control requirements. *Transportation Research Record* 736, 11-13.
[35] MORENO, A.T., FERRER, V., GARCIA, A., 2014. Evaluation of 3D coordination to maximize available stopping sight distance. *The Baltic Journal of Road and Bridge Engineering*, 9(2), 94–100.
[36] NAVIN, F.P.D., 1990. Safety Factors for Road Design, Can they be Estimated? *Journal of Transportation Research Board*, 1280, 181-189.
[37] NEHATE, G., RYS, M., 2006. 3D calculation of stopping-sight distance from GPS data. *Journal of Transportation Engineering*, ASCE, 132(9), 691-698.
[38] NORMANN, O. K., 1953. Braking Distances of Vehicles from High Speeds. *Proceedings HRB, 22*, Highway Research Board, Washington, DC. 421–436.
[39] OECD, 2006. *Speed Management, Organization for Economic Co-operation and Development*, European Conference of Ministers of Transport.
[40] PIARC (World Road Association), 2003. Road Safety Manual. PIARC, Paris, France.
[41] FGSV (Road and Transportation Research Association), 2008. *Guidelines for the Design of Motorways* [2011]. FGSV, Cologne, Germany.
[42] SARHAN, M., HASSAN, Y., 2012. Consideration of sight distance in placement of concrete barriers on horizontal curves of roads. Transportation Research Record, 1940, 9-16.
[43] STAMATIADIS, N., 2005, Context-Sensitive Design: Issues with Design Elements. *Journal of Transportation Engineering*, ASCE, 131(5), 374-378.
[44] TRANSIT, 2003. *State Highway Geometric Design Manual*, TRANSIT New Zealand, Section 2: Basic design criteria.
[45] TRANSPORTATION ASSOCIATION OF CANADA, 1999. Geometric Design Guide for Canadian Roads.
[46] WOOD, J., DONNELL, E., 2014. Stopping sight distance and horizontal sight line offsets at horizontal curves. *Transportation Research Record Journal*, 2436, 43–50.
[47] XIA, R. X., WU, D. H., HE, J., LIU, Y., & SHI, D. F., 2016. A New Model of Stopping Sight Distance of Curve Braking Based on Vehicle Dynamics. *Discrete Dynamics in Nature and Society*, 2016, Article ID 4260705, 1-8.