DESIGN CONSISTENCY EVALUATION OF RURAL HIGHWAYS

Thesis

Submitted in partial fulfillment of the requirements for the award of

DOCTOR OF PHILOSOPHY

by

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D E C L A R A T I O N

I hereby declare that the Research Thesis entitled **'DESIGN CONSISTENCY EVALUATION OF RURAL HIGHWAYS'** which is being submitted to the **NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA, SURATHKAL** in partial fulfillment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering** is a *bonafide report of the research work carried out by me*. The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

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C E R T I F I C A T E

This is to *certify* that the Research Thesis entitled "**DESIGN CONSISTENCY EVALUATION OF RURAL HIGHWAYS**" submitted by **SOWMYA N.J.**(Register Number: CV05P03) as the record of the research work carried out by her, is *accepted as the Research Thesis submission* in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy.**

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Research Guide and Professor
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Chairman - DRPC

DEDICATED TO

MY HUSBAND

SHYAMA PRASAD

AND

MY CHILDREN

POORNA AND PRATHYUSH

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ABSTRACT

Rapid growth of population and increasing economic activities are the twin factors that contribute to the tremendous increase in the vehicle population which, in turn, contributes to the growing number of road accidents. Road accidents are complex events involving a variety of factors, including highway geometry, driver behaviour, weather conditions, and speed limits. Several studies have been conducted to determine the relationship between factors associated with accidents and their effect on safety. Improving highway design consistency is considered to be an important strategy for improving traffic safety.

Geometric design consistency evaluation is a widely used method of determining sections of highways which require alignment improvement. Identifying and treating any inconsistency on a highway can significantly improve its safety performance. A critical factor in highway design is maintaining a good consistency with a road geometry that meets the driver's expectations.

Considerable research has been undertaken to explain this concept, including identifying potential consistency measures and developing models to estimate them. However, considerable amount of work has not been carried out to evaluate the geometric consistency in India. Rural roads constitute about 80 per cent of Indian road network. Therefore, rural road safety accounts for a considerable share of the total road safety problem. In Dakshina Kannada District, and even in Karnataka state, intermediate lane highways make more than 50 per cent of the principal road network. The majority of these intermediate lane rural highways are historical routes and many of them do not follow a specific design code. Therefore, road safety of intermediate lane roads is a very important issue in the development of the country. This study aims to enhance the safety of these existing highways based on an understanding of actual driving behaviour by means of field data measurements, and to verify their conformance to a consistency evaluation model.

Both horizontal and vertical alignments are the main focus of this study. The horizontal alignment of a road consists of a straight tangent followed by horizontal curve, whereas the vertical alignment of the road consists of a level section followed by a vertical curve. This study deals with developing appropriate design consistency evaluation criteria of horizontal and vertical curves using geometric, speed, and accident data of eight intermediate lane rural highways. Geometric details of a road were collected from the field and CAD (Computer Aided Design) drawings were prepared using the details of surveying. A spot speed survey was conducted for passenger cars on the approach tangent and at the beginning, middle, and end points of horizontal curves and on the approach tangent, limiting point and summit point of vertical curves. The accident details were collected for six years (from 2005 to 2010) from different police stations.

The operating speed prediction models were developed for both horizontal and vertical curves. The speed change experienced by the same driver from tangent to curve i.e. speed differential was also studied for horizontal curves, and the models were developed to predict this change.

Consistency evaluation criteria for horizontal curves and vertical curves were also developed to enhance the safety of the alignment. Alignment indices, are the another set of quantitative measures, were used to identify the inconsistencies that exist on intermediate lane rural highways. Finally, an attempt was made to develop the relationship between safety and alignment indices.

Key words: *Design consistency, operating speed, speed differentials, alignment indices*

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NOMENCLATURE

R_c	Radius of curve (m)
L_H	Length of horizontal curve (m)
Δ	Deflection angle (deg)
D	Degree of Curvature (deg)
PTL	Preceding Tangent Length (m)
PTLS	Preceding Tangent Length before speed observation point (m)
DTL	Departure Tangent Length (m)
TS	Point at start of tangent
TM	Point at mid of tangent
S	Point at start of curve
M	Point at mid of curve
E	Point at end of curve
V_{TM}	Operating speed at tangent point of curve in km/h
V_S	Operating speed at start point of horizontal curve in km/h
V_M	Operating speed at mid point of horizontal curve in km/h
V_E	Operating speed at end point of horizontal curve in km/h
G1	Approach Gradient in %
G2	Departure Gradient in %
FA	Number of Fatal Accidents
GI	Number of Grievous Injury Accidents
SI	Number of Simple Injury Accidents
TA	Total number of Accidents
e_S	Superelevation at the start of the curve in %
e_M	Superelevation at the middle of the curve in %
e_E	Superelevation at the end of the curve in %
SD_{TS}	Sight distance available at the start of tangent in m
SD_{TM}	Sight distance available at the middle of tangent in m
SD_S	Sight distance available at the start of curve in m
SD_M	Sight distance available at the middle of curve in m
W_S	Road width at start of the curve in m

W_M	Road width at middle of the curve in m
W_E	Road width at end of the curve in m
S_S	Shoulder width at start of the curve in m
S_M	Shoulder width at middle of the curve in m
S_E	Shoulder width at end of the curve in m
$(\Delta V)_{85TMM}$	Speed differential between the tangent and middle point of curve in km/h
H	Horizontal curves with tangent length more than 100m with gradient ±2%
AH	Horizontal curves with tangent length less than 100m with gradient ±2%
CC	Combined curves i.e. horizontal curves with gradient more than 2%
V	Vertical curves
TM	Point at middle of the tangent section
LP	Point at before the crest, where sight distance is limited
SP	Point at the crest or summit point
L1	Tangent length between the end of the preceding curve and the approach tangent point, in m
L2	Length of the ascending grade section up to limit point, LP or the distance between the start point of vertical curve and LP, in m
L3	Length between points SP and LP, in m
L4	Length between points AT and LP, in m
L5	Length between points AT and SP, in m
LLS	Length of grade between LP and SP, in m
LAG	Length of ascending grade up to SP, in m
K	rate of vertical curvature in m/%
L_V	Length of vertical curve in m
A	Algebraic difference of grades in %
DL_V	Deficiency in curve length in m
SD_{TP}	Calculated sight distance at tangent point in m
SD_{LP}	Calculated sight distance at limit point in m

f	Coefficient of lateral friction
ΔV_{85}	Difference in operating speed between tangent and curve or between successive elements in km/h
PRMSE	Percentage Root Mean Squared Error
R^2	Coefficient of determination or coefficient regression
R_a^2	Adjusted R^2
CCR	Curvature Change Rate in deg/km
DC	Degree of Curvature in deg/km
CL: RL	Curve Length: Roadway Length
AR	Average Radius in m
ATL	Average Tangent Length in m
RR	Ratio of maximum radius to minimum radius,
R_{min}	Minimum radius in m
R_{max}	Maximum radius in m
TL	Tangent Length in m
VCCR	Vertical curvature change rate of a highway section in deg/km
AVC	Average rate of vertical curvature of a highway section in m/%
COMBO	Combination curvature change rate of a highway section in deg/km

CHAPTER 1

INTRODUCTION

1.1 GENERAL

The rapid socio-economic development in India has resulted in tremendous growth of population and motor vehicles. On one side, the road transportation is expanding, while on the other side road accidents and fatalities are increasing alarmingly. World Health Organization (WHO) in 2004 had assessed traffic accidents as the world's ninth most important health problem. It is believed that by the year 2020, road crashes will be the third leading cause of deaths and disability. Hence, road safety is becoming a major source of concern in the present context. This chapter intends to provide a systematic and comprehensive review as guidance and provide a greater understanding of the existing body of knowledge.

1.2 ROAD NETWORK SCENARIO

One of the vital infrastructures needed for economic development and social betterment of a country is transport. The transport sector accounts for a share of 6.4 per cent of India's Gross Domestic Product (GDP), and out of this road transport alone accounts for 4.5 per cent (Kadilali and Shashikala 2009). Road transport in India is growing at a rate of 10-12 per cent per annum (Gangopadhyay 2010). India's road network, spanning across 4.69 million km, is the third-largest road network in the world, next in line only to the US and China. The country relies heavily on its robust road network that carries almost 65 per cent of freight and 80 per cent of passenger traffic (http://www.ibef.org/industry/infrastructure/roads-india.aspx). India is a developing nation hence; there is a constant demand for good quality infrastructure, transportation and services. Table 1.1 shows the statistics of Indian road network.

Table 1.1 Statistics of Indian Road Network (MORTH 2011)

Category of Road	Length in km
National Highways/Expressway	70,934
State Highways	1,54,522
Major and Other District Roads	25,77,396
Rural Roads	14,33,577

The National Highways are the backbone of the road infrastructure in India, while State Highways and Major District Roads constitute the secondary system of road infrastructure. As of February 2008, out of the total length of 7,000 km of completed highways, 14% had four or more lanes and about 59% had two lanes, while the rest (27%) of the National Highway network had single or intermediate lane (MORTH 2011).

Table 1.2 presents road statistics of Karnataka State, India and Dakshina Kannada district as on March 2010. According to the statistical data of 2010, road network in Karnataka consists of a total of 20,528 km length of State Highways out of which 9,231 km length is of single lane, 8592 km length of intermediate lane and 2,496 km length is of two lanes. Also, in Dakshina Kannada district, out of 529 km length of State Highways, 213 km length is of single lane, 299 km length of intermediate lane and only 17 km length is of two lanes (http://www.kpwd.gov.in/roads.asp - accessed on 29-04-2012).

From the statistics it is clear that both in Karnataka state and Dakshina Kannada district intermediate lane roads play an important role. Therefore, road safety of the intermediate lane highways assumes much importance in the development of the country.

Table 1.2 Road Statistics as on March 2010

Category of Road	Length of Road in km	
	Karnataka State	Dakshina Kannada district
National Highways	4,490	266
State Highways	20,528	529
Major District Roads	50,436	774
Total	75,454	1,569

(Source: Road Statistics of Karnataka State as on March 2010)

1.3 VEHICLE OWNERSHIP SCENARIO

Worldwide more than 0.8 billion motor vehicles were in use in year 2005 and this number reached around 1.0 billion in 2011. In 1998, India had just 7 vehicles per 1000 persons and presently it is 12 per thousand whereas the US has a maximum of 767 vehicles per 1,000 people and Australia has more than 600 vehicles per 1,000 people (Urban Transport Development in India Energy and Infrastructure Unit South Asia Region - Report, 2005). In India about 4 millions registered motor vehicles are added every year, while the road length has not proportionately increased over the years (Chandra and Prashanth 2004).

With rising purchasing power of average Indian, motorized vehicle ownership is growing at a fast pace and a few of the major cities have vehicle ownership at a level comparable to that of the developed countries. India had about 115 million registered motor vehicles at the end of fiscal year 2008-09. Personalized mode (constituting mainly two wheelers and passenger cars) accounted for more than four-fifths of the motor vehicle population in the country compared to their share of little over three-fifths in 1951. Further, break up of motor vehicle population reflects preponderance of two-wheelers with a share of about 72 per cent in India’s total vehicle population, followed by passenger cars (including jeeps and taxis) at 13.3 per cent and other vehicles (a heterogeneous category which includes 3 wheelers (Light Motor Vehicle (LMV)) - Passengers), trailers, tractors, etc. at 8.4 per cent (Road Transport Year Book (2007-2009)).

The number of vehicles has been growing at an average rate of around 10 per cent per annum (http://www.transportindia.in/indian_roads.asp). Developed countries like Germany (565) and USA (461) have car penetration rates (car/1000 persons) which are higher by factors of about 31 and 26 to that of China (18) and by factors of 57 and 46 to that of India (10). With raising per capita income in India this ratio is likely to see rapid surge in the coming years. In case of India (58) the penetration level of two wheelers (two wheelers / 1000 persons) is much higher compared to developed countries (Road Transport Year Book, 2009).

The number of vehicles in Karnataka has increased from 14.33 lakhs in 1990-91 to 39.96 lakhs in 2001-02 showing almost a threefold increase over the twelve years. Among the various types of vehicles plying on the roads two wheelers constitute 71.8%, followed by cars 9.5% and other vehicles 9.57% (http://www.kpwd.gov.in/roads.asp as on 31-03-2010). Total number of registered motor vehicles in Karnataka increased to 69.53 lakhs in 2009 (http://www.transport.rajasthan.gov.in/pdf%20Files/Static%20pdf/Table%209.2.pdf). The growth of motor vehicle traffic in Dakshina Kannada district is phenomenal. The number of motor vehicles increased from 2, 13, 503 in 2003 to 2, 34, 295 in the year 2004 and to 2, 58, 561 in the year 2005 showing an annual growth of around 10%. Also, the number of motor vehicles in the district has increased to 4, 29,543 at the end of March, 2011. The growth of vehicular traffic on roads has been far greater than the growth of road network. As a result, the main arteries face capacity saturation and an alarming growth in the number of accidents.

1.4 ACCIDENT SCENARIO

The pace of growth of road infrastructure in India is not commensurate with the traffic growth; as a result, road accidents are increasing in an alarming proportion. India has one per cent of world’s motor vehicle population and it accounts for nearly six per cent of world’s reported total road traffic accidents (Rajan 2006). Fatality rates are 20-30 times higher as compared to that in USA and Japan. At present, over 1, 30,000 persons are killed on the roads every year, and the economic loss due to road accidents is estimated to be over Rs.75, 000 crores per annum. It is shocking to note from the WHO report that India has the highest number of road accident deaths, while compared to other countries in the world including the more populous China.

During the period 2000-2007, road fatalities in India increased by 45.2%, whereas in other countries there was a decreasing trend varying from 13.6% to 42.8%. (Srinivasan 2011). The number of persons injured per lakh of population indicates a more than threefold increase (from 13 in 1970 to 45 in 2007). Similarly, a person killed per lakh of population indicates a more than threefold jump (from 2.7 in 1970 to 10 in 2007) (MORTH 2009).

Due to rapid infrastructural development, Karnataka has earned the distinction of being the State with the third highest accident rates in India after Tamil Nadu and Maharashtra. Statistics with the National Crime Records Bureau indicate that at least 23 people die every day in Karnataka. Statistics also reveal that in the year 2003 there were 37,658 accidents and in the year 2009, the figure rose to 45,190. (http://www.transport.rajasthan.gov.in/pdf%20files/static%20pdf/table%209.2.pdf). Statistical data of Dakshina Kannada district reveal that in the year 2003 there were 1,740 accidents which increased to 2,028 in the year 2005 but decreased to 1,796 in the year 2010. The number of persons killed in accidents increased from 242 in 2003 to 280 in 2010, showing a continuous increase in fatality. Hence, it is necessary to consider road safety measures in order to reduce the number of crash incidents. The main objective of improving road safety is to prevent accidents in the future or at least to reduce the severity of such accidents.

In India, as the funds allocated for traffic safety are meagre, it is necessary to spend the available resources as judiciously as possible. In this study, an attempt is made to develop design consistency evaluation criteria in order to identify inconsistent locations. These criteria can also be adopted in the highway design in order to improve the inconsistent locations, thereby preventing accidents in the future, or at least reducing the severity of such accidents.

1.5 RURAL ROADS

Roads in India are the most preferred mode of transportation. Easy availability, adaptability to individual needs and cost savings are some of the factors working in favour of road transport. Roads are considered to be one of the most cost-effective and preferred modes of transportation. Rural roads are major and minor roads outside urban areas and with a population of less than 10,000 (Department for Transport, U.K.) (http://www.communities.gov.uk/publications/planningandbuilding/urbanrural). Rural roads form one of the basic infrastructures in achieving the objective of integrated rural development. Rural transportation network will give shape to the living environment of villagers and roads of rural transportation are the connectivity elements in the country. It helps to bring about national integration as well as provide for country's overall socioeconomic development.

India has essentially a rural-oriented economy with 74 per cent of its population living in its villages scattered all over the country. Rural roads constitute about 80 per cent of Indian Road Network (MORTH 2011). Rural road safety accounts for a considerable share of the total road safety problem (Cafiso et al. 2005). In the United States more than 56% of road fatalities occurred on rural roads. The risk of being killed on rural roads per km driven is generally higher than on urban roads and 4-6 times higher than on motorways (Korzeniewski 2009). Also, more than 30% of the total fatalities on rural highways can be attributed to the accidents that take place on curved sections than on straight segments (Stewart and Christopher 1990, Gibreel et al. 1999). Thus, curved section and the corresponding transition sections represent the most critical locations while considering measures for improvement of highway safety. The improvement of rural roads will reduce the transport cost and increase the efficiency in the movement of passengers and goods by providing uninterrupted traffic.

Due to limited resources and constraints in Karnataka state, majority of State Highways and even National Highways are of intermediate lane. Therefore, safety of intermediate lane highways acquires major importance. On account of budgetary constraints, the high cost of upgrading one section of a highway to a rigorous standard might affect other improvement projects. Also, changes in the design of road and traffic environment affect road safety performance of highways. Thus several approaches have arisen for an assessment of geometric design called design consistency. Consistency evaluation of highways gives high level of riding comfort and maximum benefits in terms of vehicle operating cost and savings in accident cost.

1.6 IMPORTANCE OF GEOMETRIC DESIGN CONSISTENCY

One of the main reasons for accident occurrence is lack of geometric design consistency (Gibreel et al. 2001). Several studies have been conducted to determine the relationship between factors associated with accidents and accident characteristics. These studies strongly suggest that the accident occurrences and severity are greatly influenced by road geometrics, traffic volume, and speed characteristics. Dangerous spots are those which are characterized by criticalities in geometrical elements, either in plan or in profile or both. Sometimes the dangerous spots can be accident spots also if they permit larger variability in the response of the drivers (Nagaraj et al. 1990). Nadaf (1999) found that 74% of accidents occur due to driver's fault and 3% of accidents are due to bad roads. Since most drivers travel as fast as they feel comfortable and slow down only where necessary, rural highways with lower design speed exhibit uneven operating speed profiles, i.e. drivers accelerate to their desired speed on tangents and gentle curves and decelerate only on sharper curves. The inconsistencies that exist on a roadway can produce a sudden change in the characteristics of the roadway, which can surprise motorists and lead to a speed error. These speed errors result in critical driving manoeuvres for motorists and an unfavourable level of accident risks. Hence proper design and construction of physical road features facilitate in achieving maximum safety to traffic.

The objective is to develop such road conditions which can forgive driver's mistake and allow recovery or minimum severity of accidents and at the same time, guide drivers in such a way that there is little possibility of drivers making mistakes (Shaheem and Gupta 2005). It is well-recognized by transportation engineers that highway design consistency is a very important issue related to road safety. A consistent alignment would ensure that most drivers would be able to operate safely at their desired speed along the entire alignment. Evaluating the design consistency of existing rural highways has been believed to be an efficient way to identify consistency problems in the design stage of new highways and therefore, to avoid or eliminate potential inconsistent designs. Hence, the importance of design consistency and its significant contribution to road safety can be justified with an understanding of the driver-vehicle-roadway interaction.

1.7 MEASURES OF DESIGN CONSISTENCY

Most of the research on design consistency has focused on identifying quantitative measures for design consistency evaluation and developing models to estimate them. Geometric design consistency is categorized into three main areas: (1) Speed considerations, (2) Safety considerations, and (3) Performance considerations (Gibreel et al. 1999). Speed considerations address the effects of different design parameters on the prediction of operating speed that effects the evaluation of design consistency of different road elements. The difference between operating speed and design speed is a good indicator of any inconsistency at a single element, while the speed reduction experienced by drivers while travelling from one element to the next indicates the consistency between two successive elements. Safety considerations address the effect of design parameters on highway safety. Special attention is given to the effects of side friction and super elevation design on vehicle stability and to the effect of low-cost improvements on highway safety. Performance considerations address the effect of design parameters on the driver workload and driver anticipation. They also include other aspects that affect driver performance such as the aesthetics of highway alignment and interchange design.

In a study Polus and Dagan (1987), Anderson et al. (1999) and Fitzpatrick et al. (2000a) used alignment indices for consistency evaluation**.** Alignment indices are the quantitative measures of the general character of a roadway alignment. Large increase or decrease in the values of alignment indices for successive roadway segments is an indication of geometric design consistency. Fitzpatrick et al. (2000b) used speed distribution measures as another method of design consistency to identify potential problems of individual features on specific highway. Significant changes in speed distributions may suggest that design inconsistencies are present at that alignment feature.

The most common approach in the United States to ensure consistency in the design of highway has been the design speed concept. In the 1980s' Australian research found 85th percentile speeds faster than design speeds on curves with design speeds less than 90 km/h and these findings were implemented in design procedures and used to check speed consistency between successive elements (Mclean 1981, Fitzpatrick et al. 2000b). Lamm et al. in the 1980s (Lamm et al. 1988), and Krammes et al. in the 1990s (Krammes et al.1995a, b) developed speed-based consistency evaluation procedures for the United States' use. Krammes et al. (1995b) concluded from his study that the model framework adopted by Lamm was sound in concept, appropriate in sophistication, and reasonable in data requirements in two lane rural highways.

Xiong et al. (2005) also used operating speed concept which showed good results for freeway design consistency evaluation in China. According to Hong and Oguchi (2005) in Japan and Perco (2006) in Italy, operating speed is the simple method of design consistency evaluation. In India not much literature is available on geometric design consistency evaluation. In a study Kadiyali et al. (1981) considered rural stretch of road near Delhi to predict mean free speed of different vehicles using the geometric variables. During the literature survey it was observed that in India, most of the works done on road safety, highway capacity, and congestion reduction were based on speed data.

Safety considerations address the effects of side friction and super elevation design on vehicle stability. Germany in late 1980s (Lamm et al. 1988,1991) and the United States in the 1990s used the range of safety consideration criteria developed for evaluating as good, fair and poor design levels. Nicholson (1998) in Australia used margin of safety as design consistency of horizontal alignments on rural highways. The consistency of safety margin can be achieved when its standard deviation is small.

Design consistency based on performance considerations was developed considering comfortable, efficient, and safe traffic operation conditions. Performance considerations address the effect of design parameters on the driver workload and driver anticipation. It is well-known that the most valuable tool for evaluating geometric design consistency is actual collision experience, and that too high or too low workload and sudden changes of workload are usually the reason of road collisions (Messer, 1980 and Fitzpatrick et al. 2000a). However, researches in these two fields have never been boomed since it is very hard to evaluate design consistency accurately based on safety and workload due to difficulties in data collection and involvement of human factors. Therefore, the use of design consistency based on performance considerations is much more limited than operating speed (Gibreel et al. 1999).

Polus and Dagan (1987), Anderson et al. (1999), and Fitzpatrick et al. (2000a) suggested several alignment indices to quantify the general character of the alignment of a roadway. The hypothesis used in their study that alignment indices (representing the character of the preceding alignment) influence drivers' desired speeds was motivated by experiences in Germany, and England. In Germany, curvature change rate was used to estimate 85th percentile speeds along a segment of roadway (Lamm et al. 1988). In England, the average speeds along a roadway are estimated based on an alignment constraint (essentially, an alignment index) and a layout constraint (that represents the cross-section of the roadway) (Fitzpatrick et al. 2000a). The findings of a 1990s' FHWA research project indicated that, although alignment indices by themselves and combinations of alignment indices explains variation in the two lane rural highways, they suggested to use alignment indices to evaluate design consistency along with other design consistency methods to check its accuracy (Fitzpatrick et al. 2000b, NCHRP Report 502).

The United States in late 1990s' used speed distribution measures as another method of design consistency to identify potential problems of individual features on a specific highway. These results indicate that speed variance is not an appropriate method of measure of design consistency for horizontal curves on two-lane rural highways (Collins et al. 1999, Fitzpatrick et al. 2000b and NCHRP, Report 502).

Therefore, the most common and simple approach under geometric design consistency is observed to be based on operating speed. Hence, in the present work, it is planned to use speed-based design consistency measure to evaluate the consistency of both horizontal and vertical curves. Alignment indices are the non-speed based method that quantitatively measures the general character of a highway alignment. They are easy for the designers to use, understand, and explain. Therefore, in this study an attempt is made to evaluate the design consistency of intermediate lane highways using alignment indices.

1.8 NEED FOR THE STUDY

Design consistency considerations have now been shown to have a direct relationship to safety of horizontal alignment on rural two lane highways. On review of literature, it was found that a number of geometric design consistency related studies were carried out in other countries, but, in India no such work has been reported on rural highways. In India the applicability of consistency criteria developed in the other countries is questionable because the road geometric characteristics, traffic condition, and driver behaviour are totally different. Also, several models have been developed to predict the operating speed at the tangent and curved sections, but, in most cases, the model format, explanatory variables, and regression coefficients are different from one model to the other. Hence it is necessary to develop design consistency methodology for Indian conditions.

Therefore, further development of the design consistency concept is needed to extend the design consistency concept to other roadway types. In a developing country like India, due to difficult geographical condition and scarcity of funds available, many of State highways and even some of National highways are of intermediate lane. Also, it is not possible to improve the alignment conditions throughout India at a time because it requires a lot of infrastructural facility. Hence, it is necessary to evaluate the design consistency of existing intermediate lane rural highways and then that corrective measure in the field which will reduce the frequency of accidents. Dakshina Kannada is one of the most commercially and historically important districts of Karnataka state. In Dakshina Kannada (D.K.) district and even in Karnataka state, majority of highways are of intermediate lane rural roads (pavement width of 5.5 m). On account of rapid socioeconomic development in D.K. district, it stands second in Karnataka state among the developed districts and, owing to increase in human and vehicle population in the district, there is an alarming rise in road accidents. Hence, it is necessary to consider design consistency measures to avoid accidents in the future or at least reduce the severity of accidents.

The key issues for creating a good road network in the district would relate to the arrangements to be made that would help remove the existing deficiencies on a sustainable basis through a proper system of operation and maintenance. The improvement of highways will reduce the transport cost and increase the efficiency in the movement of passengers and goods by providing uninterrupted traffic within the district.

1.9 OBJECTIVES

The main objective of this study is to evaluate the design consistency of the selected curved stretches using speed based measure, such as operating speed and speed differential and non-speed based measure, such as alignment indices. Considering the main aim as a key point, the objectives of the study can be stated specifically as follows:

- To establish a relation between speed and independent geometric variables by developing of speed prediction models for horizontal and vertical curves of rural highways
- Development of design consistency evaluation criteria for horizontal curves and vertical curves of rural highways
- Evaluation of highway geometric design consistency using alignment indices

1.10 SCOPE

In this study, several efforts are made to predict the operating speed and speed differential models for horizontal curves and to predict the operating speed models for vertical curves. The main benefit of these speed prediction models are: 1) the operating speeds and 85th speed differential can be estimated for a highway alignment during the design process and necessary corrections can be made in the design. 2) to identify the inconsistency locations of an existing section, by knowing the geometric details of a particular location.

Alignment indices are the non-speed-based design consistency evaluation method used to strengthen the evaluation which can be applied in the field when it is not possible to evaluate the consistency by speed.

1.11 STRUCTURE OF THESIS

This thesis consists of seven chapters:

Chapter 1 **Introduction** presents an introduction outlining accidents, road network and vehicle ownership scenario, importance of design consistency and measures of consistency evaluation.

Chapter 2 **Literature review** provides an extensive literature on design consistency and its methods.

Chapter 3 **Consistency Evaluation of Horizontal Curves** describes the data collection and the methodology used to develop the relationship between speed and geometric variables of horizontal curve. It also describes the consistency evaluation criteria developed for horizontal curves.

Chapter 4 **Consistency Evaluation of Vertical Curves** describes the data and the methodology used to develop the relationship between speed and geometric variables of vertical curve. Consistency evaluation criteria for vertical curves are also proposed in this Chapter.

Chapter 5 **Consistency Evaluation by Alignment Indices** provides a detailed study of design consistency evaluation of alignment using Alignment Indices.

Chapter 6 **Summary, Conclusions, Future scope** summarizes the study effort and provides conclusions. It also gives some recommendations for future research.

The **References** are included at the end of this thesis.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The geometric design of the roads has been studied for many years. Many researches have been carried out on several elements of geometric design such as horizontal alignment and vertical alignment. Most of these researches had been widely conducted in the United States, Europe, Japan and other developing countries. This chapter is designed to provide a systematic and comprehensive review as guidance and provide a greater understanding of the existing body of knowledge. The current chapter also provides an extensive literature review on geometric design consistency and its relationship to road safety. Emphasis is placed on research on methods of design consistency evaluation which are discussed elaborately.

2.2 HIGHWAY GEOMETRIC DESIGN

The geometric design of a highway deals with the dimensions and layout of visible features of the highway such as alignment, sight distances and intersections (Khanna and Justo 2001). The desire of every road user is to reach his destination in the shortest possible time and with least inconvenience. Therefore, speed, safety, and comfort are the three important factors which control the design features of geometric layout. The good geometric design means providing an appropriate level of mobility and land use access for drivers and pedestrians while maintaining a high degree of safety (Wooldridge et al. 2003).

Highway design standards are necessary to maintain uniformly across the country for the specific classification. These standards are developed based on combination of empirical formulae, professional experience, and judgment. Applying the highest geometric design standards can maximize highway safety. However, limited resources and constraints resulting from physical, right-of-way, and environmental features often restrict the highway designer’s ability to develop geometric designs that exceed minimum design standards. In India, Indian Roads Congress (IRC) has given various design standards for road depending upon the topography, locality, and the type and intensity of traffic. Changes in the design of alignment and traffic environment affect road safety performance of highways. Thus, several approaches have arisen for an assessment of geometric design called design consistency.

2.3 DESIGN CONSISTENCY CONCEPT

The concept of design consistency has been gaining greater acceptance in all over the world. Considerable research work is being carried out and steps have already been taken to incorporate this concept into the design practices in developed countries. However, a standard procedure that can be followed by designers to evaluate the design consistency of new or existing alignments is still lacking. The main objective of the design consistency concept is to develop the relationship between design consistency measures and safety. The well-balanced design policy especially considers the driver's perception and behaviour in the evaluation of highway geometric design (Hong and Oguchi 2005).

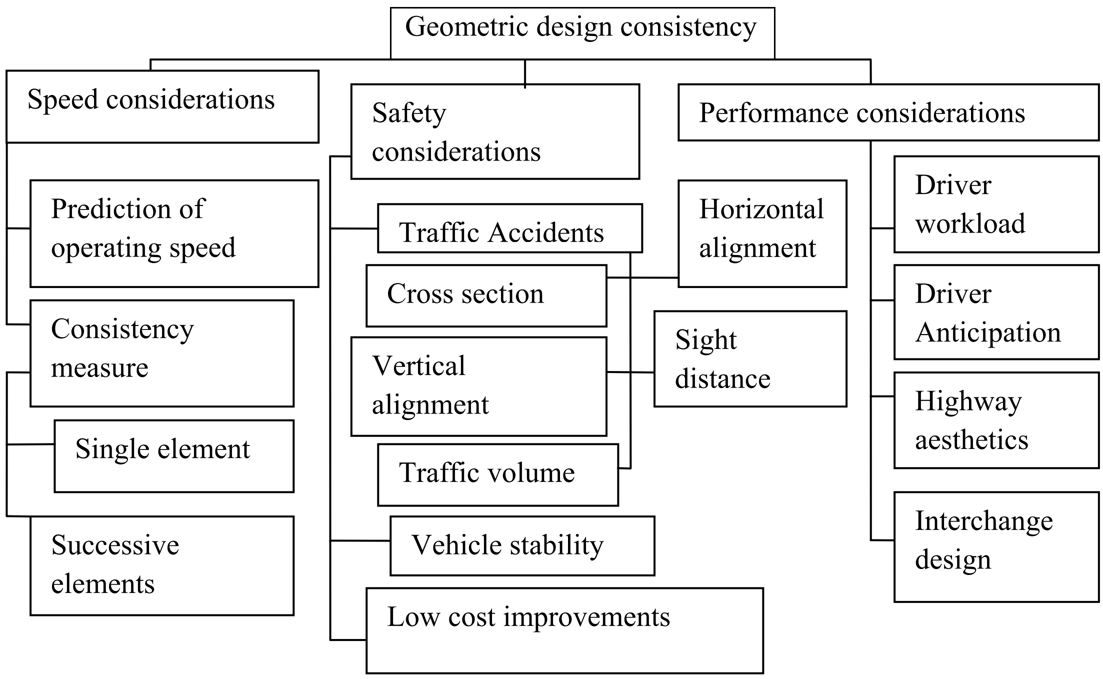
A design inconsistency in a roadway segment usually results from geometric features that vary significantly and, therefore, may cause drivers to make speed errors or unsafe driving manoeuvres leading to higher collision risk. Therefore, geometric design consistency is emerging as an important component in highway design (Glennon and Harwood 1978). An inconsistency in design can be defined as a geometric feature or combination of adjacent features that have such unexpectedly high driver workload that motorists may be surprised and possibly drive in an unsafe manner (Messer 1980). Design consistency is also defined as the conformance of the highway geometry and operational features with driver expectancy (Nicholson 1998). Gibreel et al. (1999) referred to a consistent highway design as one that ensures that successive geometric elements are coordinated in a manner to produce harmonious driver performance without surprising events. Also, design consistency implies that the design or geometry of a road does not violate either the expectation of the motorist or the ability of the motorist to guide and control a vehicle in a safe manner (Fitzpatrick et al. 2000a).

Consistent roadway design is important because it is believed that motorists make fewer errors at geometric features, which conform to their expectations than at features that violate their expectancies (Fitzpatrick et al. 2000b). Driver expectancies are based on their experiences in the immediate past and over their driving careers (Wooldridge et al. 2003). Some researchers have explicitly noted that treating any inconsistency in a highway alignment can significantly improve its safety performance (Joanne and Sayed 2004). Thus, evaluating design consistency and identifying any inconsistencies during the design stage of newly designed highways can significantly improve the safety of the highway network.

2.4 DESIGN CONSISTENCY EVALUATION

A critical factor in highway design is maintaining a good consistency with a road geometry that fits the drivers' expectations. The United States and other countries have proposed several methodologies that account for problems associated with design consistency. The design consistency methods are categorized into two ways i.e., designer side methods that focus on operating speed and user side methods that focus on driver mental workload (Nicholson 1998). Fitzpatrick et al. (2000b) grouped design consistency evaluation into four areas like vehicle operation-based consistency, road geometric-based consistency, driver workload, and consistency checklists. Vehicle operation-based consistency measure focuses on operating speed, road geometric-based consistency focuses on roadway geometry (alignment indices), and consistency checklists focus on examining design features for possible expectancy violations. Based on literature survey work on geometric design consistency, Gibreel et al. (1999) categorized design consistency into three main areas: (1) Speed considerations, (2) Safety considerations, and (3) Performance considerations as presented in Fig.2.1. Speed considerations address the effects of different design parameters on the prediction of operating speed that effects the evaluation of design consistency of different road elements. Safety considerations address the effect of the design parameters on highway safety. Special attention is given to the effects of side friction and superelevation design on vehicle stability and to the effect of low-cost improvements on highway safety. Performance considerations address the effect of design parameters on driver workload and driver anticipation. They also include other aspects that affect driver performance such as the aesthetics of highway alignments and interchange design.

Alignment indices are another quantitative measure of geometric design consistency of a roadway's alignment. Large increase or decrease in the values of alignment indices for successive roadway segments is an indication of geometric design inconsistency (Polus and Dagan 1987, Anderson et al. 1999 and Fitzpatrick et al. 2000a).



`Fig.2.1 Main areas of geometric design consistency (Gibreel et al. 1999)

Garber and Gadiraju (1989), Collins et al. (1999) and Fitzpatrick et al. (2000b) have used speed distribution measures as another method of design consistency to identify potential problems of individual features on specific highway. Significant changes in speed distributions may suggest that design inconsistencies are present at that alignment feature.

2.5 DESIGN CONSISTENCY BASED ON SPEED CONSIDERATIONS

The change in speed of vehicles is a visible indicator of inconsistency in geometric design (Nicholson 1998) because a large portion of collisions have been attributed to improper speed adaptation (Al-Masaeid et al. 1995). "Speed thrills, but kills" is all the more relevant today with the modern technology motor vehicles, but it remains ignored by the community and also by authorities (Srinivasan 2011). Considering the alarming nature of this problem, there is an urgent need to develop a national framework for determining appropriate vehicle speeds on all roads, and steps should be taken to enforce them.

Speed is an important factor that is usually considered on the route selection or the choice of transportation mode. The factors which influence the speed selected by drivers on individual curves are radius of curves, sight distance, roughness, super elevation, transition curve, widening of pavement on curves, width of shoulders and type of shoulders and delineation of lanes (Kadiyali et al. 1981)**.** Speed contributes to about 40% of traffic crashes and deaths. Also, a 5% increase in average speed leads to an approximately 10% increase in accidents that cause injuries and 20% increase in fatal accidents (Srinivasan 2011). With increase of every 1 mile/hr speed above design speed associated with about 10-14% increased risk of fatal accidents (Taylor 2001). Inappropriate speeds on rural roads and excess of the speed limit on urban roads contribute to the growing evidence for accidents. Most consistency concepts today deal with the variation in the speeds of vehicles on segments of highways (Lamm et al. 1996, Al-Masaeid et al. 1995). This speed variation is affected mainly by longitudinal elements of the horizontal alignment and vertical element. As horizontal and/or vertical elements of highway features change, so do the speeds.

Two-speed approaches to evaluate design consistency on two lane rural highways include design speed and operating speed (Fitzpatrick et al. 2000a). According to AASHTO Green Book the design speed is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the features of the highway govern. The assumed design speed should be a logical one with respect to the topography, the adjacent land use, and the functional classification of highway. U.S Bureau of Public Road Engineers recommended design speed valves as equivalent to the anticipated 95^{th} or 98^{th} percentile operating speed (Fitzpatrick et al. 2000a). The operating speed describes the actual speed of a group of vehicles over a certain section of roadway. According to AASHTO Green Book, operating speed is the speed at which drivers are observed operating their vehicles during free flow conditions (Anderson et al. 1995). A free flowing vehicle is defined as having 5 sec headway or 3 sec tail way (Fitzpatrick et al. 2000b). The 85th percentile of the distribution of observed speed is the most frequently used measure of operating speed (Gibreel et al. 1999).

2.5.1 Design Speed Concept

The most common approach in the United States to ensure consistency in the design of highways has been the design speed concept. The concept was developed in the 1930s' by Barnett and incorporated into AASHTO policy in the 1940s' and is currently used in the United States (Fitzpatrick et al. 2000a) and in most western European countries (Kanellaidis et al. 1990) for the determination of minimum radii of curves, superelevation and stopping sight distances. Also, in India all the geometric features are designed based on design speed recommended by IRC. The design speed concept involves the selection of a design speed based on the topography, the adjacent land use, and the functional classification of highway.

Fitzpatrick et al. (2000a) and Hassen et al. (2001) also identified the weakness of the design speed concept is that; it is used to design geometric elements within the section of a highway. Also, the geometric alignments designed based on the design speed concept may generate operating speeds that fluctuate considerably along the different road sections or are considerably higher than the design speed. For curves with speed standards of 100 km/h or more, 85th percentile speeds are generally less than the curve speed standards. While for curves of lower speed standards the reverse applies, for curves with speed standards between 40-75 km/h, the 85th percentile speeds tend to be about 12 km/h above the speed standard. Hence, design speed approach to alignment design does not hold true for design speeds of less than 90 km/h (Mclean 1981). Also, the design speed applies only to horizontal and vertical curves and not to the tangents that connect those curves (Fitzpatrick et al. 2000a, b). Hence operating speed based design consistency has been suggested and is implemented in many countries (Lamm et al. 1996, Gibreel et al. 1999).

2.5.2 Operating Speed Concept

In majority of literature, it is observed that researchers have used operating speed as a design consistency measure. The operating speed is often determined as the 85th percentile speed (V_{85}) of a sample of vehicles (Hassan 2004). Actual operating speed is affected by many factors, such as internal environments (driver condition, vehicle performance, load condition, etc.), external environments (road alignments, number of lanes, posted speed, traffic condition, sight distance, etc.), and weather conditions (dry, wet, or snowy surface, foggy scenery condition, heavy rain, etc.) (Hong and Oguchi 2005). Highway geometric design consistency is usually evaluated on the basis of operating speed profile analysis, which requires the use of operating speed models. Design inconsistencies are identified on the speed profile when there are large differentials in operating speeds between successive alignment features. Switzerland was one of the first countries to use speed profile models in the geometric design of roadways (Fitzpatrick et al. 2000a).

2.5.3 Design Consistency Criteria

Lack of speed consistency has been used to measure the deficiencies of different highway geometric designs. These indicators can assist in the design and management of safe and efficient highways for prevailing traffic conditions.

Potential design inconsistencies result from exceeding the design speed on a specified curved section or from significant difference in operating speeds on two successive sections (Lamm et al. 1988). Generally the greater the difference between the operating speed on a curve and the design speed of the curve, the greater the design inconsistency and the risk of collisions on the curve (Hassen et al. 2000). The magnitude of this variability of speeds is essentially converted to an estimate of highway consistency. European standards recommended two speed criteria (safety criterion I and safety criterion II) based on design speed and on operating speed to evaluate the design as good, fair or poor (Lamm et al. 1988, 1996, Hassan 2004). Safety criterion I deal with the design speed and observed operating speed for single element, but safety criterion II deals with the operating speed transition between successive design elements.

2.5.3.1 Single element

The safety criterion based on design speed relates to the circular curve itself i.e., the selected design speed (V_d) with the actually observed operating speed (V_{85}). Based on mean accident rates and the difference between V_d and $V_{85.}$ Lamm et al. (1988, 1996) suggested an evaluation criterion to evaluate on a single highway element as,

Good design: $V_{85}- V_d \leq 10$ km/h

Fair design: 10 km/h< $V_{85}-V_d \leq 20$ km/h

Poor design: $V_{85}- V_d > 20$ km/h

In another approach it was recommended that the difference between operating speed and design speed on a specific highway section should not exceed a maximum of 20 km/h **(**Gibreel et al. 1999**)** and 15 km/h (Fitzpatrick et al. 2000a); otherwise, the design speed should be raised or the alignment characteristics should be modified to reduce the operating speed. Fitzpatrick et al. (2003) studied the relationship between design speed, operating speed and posted speed on two lane rural highways and found that operating speed on horizontal curves was greater than design speed when design speed was less than 70 km/h and less than design speed when design speed was greater than 70 km/h. They concluded that when operating speed is higher than design speed a speed consistency condition will arise at that location. However, the design speed concept has undergone increased criticism (McLean 1981, Hirsh 1987 Fitzpatrick et al. 2000a).

2.5.3.2 Successive element

Consistency evaluation of successive elements is based on calculating the operating speed of the drivers on the tangent and curved sections and then subtracting these values and naming it as the speed reduction value. Kanellaidis (1990) suggested that a good design can be achieved when the difference between operating speed ($V_{85)}$ on the tangent and the following curve operating speed (V_{85}) does not exceed 10 km/h. This design consistency methodology is based on assumption that speed distribution on successive elements is the same. Lamm et al. (1988, 1996) suggested another criterion to evaluate design consistency between a tangent and the following curve as follows:

Good design : $\Delta V_{85} = V_{85i} - V_{85i+1} < 10$ km/h

Fair design : 10 km/h $< \Delta V_{85} \leq 20$ km/h

Poor design : $\Delta V_{85} > 20$ km/h

Where,
V_{85i} and V_{85i+1}= Operating speeds on a tangent and the following curve elements.

Lamm et al. (1996) reviewed operating speed consistency and design speed consistency and extended design consistency to multiple lanes rural and suburban road design to evaluate good, fair, and poor design levels. This methodology was based on assumption that large changes in motorist speeds between successive geometric features represent potential design inconsistencies. Large reductions in motorist speeds represent locations with high potential for traffic accidents (Anderson et al. 1999)

On the basis of this measure, highway designers and managers may accept inconsistent and potentially unsafe highway elements, as being consistent and safe and hence are not in need of any treatment. Based on change in operating speeds between the tangent and the horizontal curves, the curves can be categorized as: safe curves when the change in speeds is less than 20%, relatively safe curves when the change in speeds is between 20% and 40%, dangerous curves when the change in speeds is between 40% and 60% and very dangerous curves when the change in speeds is greater than 60% (Wooldridge et al. 2003).

Fitzpatrick et al. (2000b) identified three sets of acceleration and deceleration values for the design consistency checks as are given Table 2.1. The acceleration and deceleration between successive elements were calculated by observed operating speed.

Table 2.1 The design consistency criteria based on acceleration and deceleration

Deceleration (m/sec^2)	Design consistency	Acceleration (m/sec^2)
1 -1.48	Good design	0.54-0.89
1.48 –2	Fair design	0.89-1.25
>2	Poor design	>1.25

Design consistency evaluation based on design speed has undergone many criticisms therefore; design consistency has been suggested by operating speed concept and implemented in many countries (Lamm et al. 1996, Gibreel et al. 1999, Fitzpatrick et al. 2000a, b and Hassen et al. 2001). Highway geometric design consistency based on operating speed concept requires the use of operating speed models.

2.5.4 Operating Speed Models

Operating speed (V_{85}) is a good indicator of the level of safety on the road segment (Kanellaidis et al. 1990). Therefore, predicting V_{85} on the various components of the highway alignment is a key step in consistency evaluation. For the past several years many studies were conducted with the aim of addressing the different effects of geometric parameters on the operating speed.

2.5.4.1 Operating speed models for horizontal curves

The 85th percentile car speeds were dominantly influenced by the desired speed (V_F) pertaining to the road section and curve radius (R_c). The sight distance restrictions affect speeds, which alters the desired speed of drivers (Mclean 1981). The developed model was

$$V_{85} = 53.8 + 0.464V_F - \frac{3.26x\,10^4}{R_c} + \frac{8.5x10^4}{R_c^{\,2}} \qquad R^2 = 0.92 \qquad \text{-- (2.1)}$$

The desired speed on a particular route length was influenced by road function, type of trip purpose and length for traffic on the road, proximity to urban centres, and the overall standard for alignment. In India (CRRI, 1982) horizontal sight distance was found to affect mean speeds by 1.4 to 3.0 km/h per 100 m. Kadiyali et al. (1981) considered rural stretch of Gurgaon -Mehrauli road near Delhi to determine the effect of radius, sight distance, and roughness on free speed. They found that mean free speed of different vehicles is more dependable on radius (R_c) and sight distance (SD), and the effect of roughness was insignificant. They also found that the free speed value with R_c =500m and SD=500m corresponds to free speed values on straight section.

Lamm et al. (1988) studied operating speeds on 261 alignments in the state of New York and statistical relationship was developed between the 85th percentile speed in horizontal curves and the degree of curve, lane width, shoulder width, Curvature Change Rate (CCR), and annual average daily traffic (AADT), but degree of curve, radius of curve, and CCR were the most significant variables in predicting operating speed at horizontal curve. The developed models are:

$$V_{85} = 58.656 - 1.135D \qquad R^2=0.79 \qquad \text{-- (2.2)}$$

$$V_{85} = 94.39 - \frac{3188.57}{Rc} \qquad R^2=0.84 \qquad \text{-- (2.3)}$$

$$V_{85} = 95.77 - 0.076CCR \qquad R^2=0.79 \qquad \text{-- (2.4)}$$

The Curvature Change Rate of a single curve was calculated using the Eq.2.5.

$$CCR = \frac{63700}{L}\left(\frac{L_{C11}}{2R_c} + \frac{L_{cr}}{R_c} + \frac{L_{C12}}{2R_c}\right) \qquad \text{-- (2.5)}$$

Where,

CCR = curvature change rate (gon/km) [gon is a designation of the angular unit (1 gon = 0.9º)]

L_{cr} = length of circular curve (m).

L_{cl1} and L_{cl2} = length of spirals preceding and succeeding the circular curve (m)

R_c = radius of curve (m) and

L = total length of curve and spirals (m).

Hassan et al. (2001) observed advantage of CCR over the traditional degree of curve D, as it can consider transition curves and can be used for compound curves. The speed behaviour of drivers on rural road curves was also studied by Kanellaidis et al. (1990) in Greece. Based on 58 curve sites data, they found that operating speed tends to be better related with powers of curvature less than one than with powers greater than one. Also, they found that the operating speed is strongly related to the curvature and to the desired speed. The developed models are:

$$V_{85}=129.88-\frac{623.1}{\sqrt{R_c}} \qquad R^2=0.777 \qquad \text{-- (2.6)}$$

$$V_{85}=32.2+\frac{226.9}{R_c}-\frac{533.6}{\sqrt{R_c}}+0.8393V_F \qquad R^2=0.925 \qquad \text{-- (2.7)}$$

The changes in operating speed on horizontal curves of two lane rural highways were studied by Islam and Seneviratne (1994) at eight sites along US-89 in Utah. The study locations had degree of curvature ranging from 4° to 28° and grade was less than 5 per cent. It was found that the degree of curve is the most significant parameter in predicting operating speed on horizontal curves. It has been also found that as the difference between operating speed at PC, MC, and PT increases, speed inconsistency problems may arise gradually with the increase in the degree of curvature above 8^o. The developed models are:

$$V_{85} = 95.41-1.48D-0.012D^2 \quad \text{(Point of Curve: PC)} \quad R^2=0.99 \qquad \text{-- (2.8)}$$

$$V_{85} = 103.03-2.41D-0.029D^2 \quad \text{(Middle of Curve: MC)} \quad R^2=0.98 \qquad \text{-- (2.9)}$$

$$V_{85} = 96.11-1.07D \quad \text{(Point of tangency: PT)} \quad R^2=0.98 \qquad \text{-- (2.10)}$$

Krammes et al. (1995a, b) presented several design consistency models to evaluate design consistency for rural two lane highways. The 85^{th} percentile speed was predicted based on such independent parameters as the degree of curve, length of curve, and the deflection angle. The developed models are:

$$V_{85}=103.66 - 1.95D \qquad R^2= 0.80 \qquad \text{-- (2.11)}$$

$$V_{85}=102.45 - 1.57D + 0.0037L - 0.1\Delta \qquad R^2= 0.82 \qquad \text{-- (2.12)}$$

$$V_{85}=41.62 - 1.29D + 0.0049L - 0.12\Delta +0.95V_T \qquad R^2= 0.90 \qquad \text{-- (2.13)}$$

From the above model (Eq.2.13), it is also observed that the 85^{th} percentile speed is also dependent on tangent operating speed (V_T in km/h). McFadden and Elefteriadou

(1997) used the bootstrapping statistical technique to sample, formulate, and validate the operating speed prediction models that were previously proposed by Krammes et al. (1995a, b). In all three models tested, they found no significant difference between the 85th percentile speed predicted by the model and the observed 85th percentile speed. The developed two operating speed-prediction models, based on the data from 78 curved sections, are:

$$V_{85}=104.61 - 1.90D \qquad R^2= 0.74 \qquad \text{-- (2.14)}$$

$$V_{85}=54.59 - 1.5D + 0.0006L - 0.12\Delta +0.81V_T \qquad R^2= 0.86 \qquad \text{-- (2.15)}$$

In the study Anderson et al. (1999) found that the grades and stopping sight distance have an effect on operating speeds. Polus et al. (2000) developed several nonlinear models for estimating operating speed on tangent sections of two-lane highways. The independent variables were the length of the tangent section and the radii of the curves prior to and after the tangent section. For example, the model for the fourth group had the following form when the tangent length was greater than 1,000 m and the radius could assume any value. Then,

$$V_{85T}=105-22.953e^{-0.00012\,(GM)} \qquad R^2=0.84 \qquad \text{-- (2.16)}$$

Where,

V_{85T} = 85th percentile operating speed on the tangent km/h

GM=Geometric Module given as

$$GM=0.01L_T\,(R_1.R_2)^{0.5} \qquad \text{-- (2.17)}$$

Where,

L_T =length of the tangent (m) and

R_1 and R_2= adjacent curve radii (m).

Abidin and Adnan (2008) analysed the operating speed behaviour at the vertical and horizontal alignment and concluded that the operating speed of the vehicle depends on the radius, gradient, lane width, and flow rate. The models can be summarised as in below equations:

For Upgrade Horizontal Curves:

$$V_{85} = -42.8 + 0.929\, R_c - 0.00468\, Q \qquad R^2=0.927 \qquad \text{-- (2.18)}$$

and

For Downgrade Horizontal Curves:

$$V_{85} = 46.5 - 0.247\, R_c - 1.02\, G - 0.00172\, Q$$

R^2=N/A (reported that the R^2 values of the equations are more than 50%), -- (2.19)

Where,

V_{85} = Operating Speed in km/h,

R_c = Radius,

Q = Flow Rate, and

G = Gradient

The units of the variables used in the model are not given by the researchers.

Ottesen and Krammes (2000) developed an evaluation model to check the speed consistency for rural alignments with design speeds of less than 100 km/h. Two curve speed models were developed that include two curve characteristics (i.e., degree of curve, D and length of curvature, L and the operating speed on the approach tangent as explanatory variables and are:

$$V_{85} = 102.44-1.57D+ 0.012L -0.01DL \qquad R^2=0.81 \qquad \text{-- (2.20)}$$

$$V_{85} = 41.62-1.29D-0.0049L-0.12DL + 0.95V_T \qquad R^2=0.90 \qquad \text{-- (2.21)}$$

The model generated was unsuccessful in determining the operating speed on straight sections; therefore, an average operating speed of 97.9 km/h was assumed along horizontal tangents. They concluded that the 85th percentile speeds on curves with degree of curvature ≤ 4° do not differ significantly from 85th percentile speeds on long tangents. However, Ottesen and Krammes developed a speed profile model in which it is assumed that the constant term in their statistical models (i.e., the predicted 85th percentile speed when the degree of curvature and length of curve are zero) represented the tangent approach speed.

Misaghi and Hassan (2005) recommended a speed of 103.0 km/h and 95.7 km/h as the 85th percentile speeds on independent and non-independent approach tangents respectively. To evaluate the actual highway geometric design Hong and Oguchi (2005) proposed a concept of 'reversely calculated speed' which includes the operating speed consistency referred to geometric conditions. They found that the horizontal radius at any point is approximately in proportion to the square of reversely calculated speed. The operation speed on wet surface was found to be significantly slower than that on dry surface. The operating speeds on dry surface are found to be affected by curvature of horizontal radius in case of high curvature, and also affected by vertical grade in case of steep uphill. But the effects of those geometric design elements are not linear. Perco (2006) developed a model at the midpoint curve and found that operating speed at the midpoint curve on two lane rural highways (50 m< R< 840 m) is dependent on radius of curve.

$$V_{85} = \frac{115}{1+\frac{75.6375}{R_c}} \qquad R^2=0.8 \qquad \text{-- (2.22)}$$

Figueroa and Tarko (2007) observed a large variability in curves with radius smaller than 518 m. They observed that the speeds on flatter curves were more dependent on the cross sectional dimensions and other road elements than curve design. The model used to estimate the 85th percentile speeds on curves in mi/h is:

$$V_{85}=51.973+0.003SD-2.639RES-2.296D+7.748e-0.624e^2 \quad R^2=0.824 \qquad \text{-- (2.23)}$$

Their results showed that 66% of deceleration transition length occurs on tangent prior to the curve and that the mean deceleration rate is 0.732 m/sec^2 and also, 72% of the acceleration transition length occurs on the tangent following the curve and that the mean acceleration rate is 0.488 m/sec^2. The developed models are:

$$V_d=1.097(V_t-0.6553\,(V_t-V_c)+0.03299\,l_d) \qquad R^2=0.840 \qquad \text{-- (2.24)}$$

$$V_a=1.097\,(V_t-0.7164\,(V_t-V_c)+0.02211\,l_a) \qquad R^2=0.876 \qquad \text{-- (2.25)}$$

Where,

SD = Sight distance in m

e = Superelevation rate (%)

D =Degree of curve (degree/100ft)

RES = Residential development indicator variable,

=1, if segment has 10 or more residential driveways per mile

= 0, otherwise.

V_d =Speed on acceleration tangent section in km/h

V_a = Speed on deceleration tangent section in km/h

V_t = Speed on the tangent in ft/sec

V_c = Speed on the curve in ft/sec

l_a = Length of acceleration in ft

l_d = Length of deceleration in ft

2.5.4.2 Speed reduction models for horizontal curves

Al-Masaeid et al. (1995) considered the speed reduction between tangent and curve as the inconsistency measure of a section. Speed reduction was found to be affected by the degree of curve, length of vertical curve within the horizontal curve, gradient, and pavement condition. The speed on the tangent was found to be affected mainly by the tangent length. The developed models of operating speed reduction between a tangent and the following curve are as follows:

$$\Delta V_{85} = 3.30 + 1.58D \qquad R^2 = 0.62 \qquad \text{-- (2.26)}$$

$$\Delta V_{85} = 1.84 + 1.39D + 4.09P + 0.07G^2 \qquad R^2 = 0.77 \qquad \text{-- (2.27)}$$

$$\Delta V_{85} = 1.45 + 1.55D + 4P + 0.00004L_v^2 \qquad R^2 = 0.76 \qquad \text{-- (2.28)}$$

Where,

ΔV_{85} = operating speed reduction between tangent and curve (km/h)

D= degree of curve in degrees

P = pavement condition

(for present serviceability rating ≥3, P = 0, otherwise P = 1)

G = gradient (%) and

L_v = length of vertical curve within the horizontal curve (m).

Anderson and Krammes (2000) estimated the reduction in 85th percentile speeds from the approach tangent to the midpoint of the following curve. They found that a statistically significant relationship existed between mean speed reduction and mean accident rate and the sites with higher speed reductions showed higher accident rates.

Fitzpatrick and Collins (2000) predicted minimum speed of 60 km/h on a horizontal curve with radius less than 100 m and suggested to use 100 km/h as the tangent speed. Hassan et al. (2000) proposed a three-dimensional operating speed models with sight distance and vehicle dynamics requirements. Based on field observations, 102 km/h was used as an operating speed on tangents and applied to predict operating speed on curves.

The methodology of reduction in 85th percentile speeds from the approach tangent to the midpoint of the following curve is based on assumptions that speed distribution on the successive elements is the same (McFadden and Elefteriadou 2000). Actually, the speed distributions on successive elements are different and each driver responds differently to the horizontal curve based on his/her desirable tangent speed and the actual side friction factor (Hirsh 1987). Thus potentially hazardous locations are not identified precisely by the speed reduction methodology. Hirsh thus summarised that it is preferable to obtain and analyse the speed distribution of individual vehicles between successive elements than speed change by comparing between the 85th percentiles of the two distributions. McFadden and Elefteriadou (2000) tried to validate Hirsh's hypothesis through the analysis of field data and introduced a new parameter 85th percentile of maximum speed reduction called 85MSR. This reflects the 85th percentile maximum speed reduction between two successive highway elements as experienced by the same vehicle or driver. They found that the speed differential captured by the new measure is an average about two times greater than the speed differential obtained from conventional ΔV_{85} measure. They also found that 85MSR between successive elements is mainly dependent on inverse of radius and length of tangent (L_T). The developed models are:

$$85MSR = -14.90 + 0.144\, V_{85T} - \frac{954.55}{R_c} + 0.0153\, L_T \qquad R^2=0.712 \qquad \text{-- (2.31)}$$

$$85MSR = -0.812 + \frac{998.19}{R_c} + 0.017\, L_T \qquad R^2=0.603 \qquad \text{-- (2.32)}$$

It can be also observed that 85MSR increases as tangent speed increases. The inconsistency locations were identified based on maximum speed reductions. Misaghi and Hassan (2005) also observed a weak relation between the traditional operating speed and speed differential model and found that ΔV_{85} underestimates the speed differential by 7.55 km/h compared with 85th percentile speed reduction between two successive highway elements ($\Delta_{85}V$), as experienced by the same vehicle or driver. Also, they found statistically significant relationship between the speed at the approach or departure tangent and the geometric or traffic features of the section. The developed models are:

$$V_{85MC} = 91.85 + 9.81x10^3R_c \qquad R^2= 0.46 \qquad - (2.33)$$

$$V_{85MC} = 94.30 + 8.67x10^3 R_c^{\ 2} \qquad R^2= 0.52 \qquad - (2.34)$$

$$\Delta_{85}V = -83.63 +0.93 V_T + e^{-8.93+ 3507.10/Rc} \qquad R^2= 0.64 \qquad - (2.35)$$

$$\Delta_{85}V= -198.74+21.42\sqrt{V_T}+0.11\Delta-4.55SW-5.36(curve\text{-}dir)+1.3 G+4.22(drv\_flag) \qquad R^2= 0.89 \qquad -- (2.36)$$

Where:

G= vertical grade (%)
curve-dir = curve direction(right turn: curve-dir=1,Left turn: curve-dir=0)
drv_flag= driveway flag
(intersection on curve: drv_flag=1,otherwise drv_flag=0)

Park and Saccomanno (2006) adopted McFadden and Elefteriadou's speed measure 85MSR, based on video-taped data from South Korea. They found that 85MSR measure is more flexible than conventional ΔV_{85} measure for estimating speed differential between successive highway elements because the 85MSR does not require a strong independency assumption for speeds established by vehicles in these elements. Also, in their study, the estimated results of 85MSR were found to be approximately 1.6 times larger than the estimated results for ΔV_{85}. Duong et al. (2010) concluded that microscopic simulated 85MSR value provides more accurate values than 85MSR values derived from statistical-based models of tangent-curve segments by McFadden and Elefteriadou, and Park and Saccomanno. They observed that there is a direct relationship between 85MSR and ΔV_{85} measures and ΔV_{85} measure is shown to provide lower speed consistency values than the 85MSR measure. The analysis showed that the deceleration rate was significantly correlated with the curve radius and also, the average rate of acceleration and deceleration under operation depends on the driver's characteristics and that maximum rates are controlled by the vehicle performance.

Acceleration and deceleration rates may be influenced by the same variables that influence 85th percentile speeds on curves (Collins and Krammes 1995). In a study by Katti and Raghavachari (1986) found that acceleration rate of 1m/sec^2 can be assumed for the fast modes under low traffic volume conditions, and deceleration rate of 2.4 m/sec^2 to 2.7 m/sec^2 can be considered as comfortable for the automobiles. An experimental investigation conducted by Collins and Krammes (1995) found that deceleration and acceleration occur within curves and speeds are not constant throughout a curve, and that the observed average acceleration rate for vehicles departing a curve is consistently lower than 0.85 m/sec^2. Anderson et al. (1999) concluded that most of the speed reduction will occur on the approaches to the curves rather than on the curve itself. Hassan et al. (2000) assumed that the speed transition occurs entirely on the approach tangents and applied the acceleration rate of 0.85 m/sec^2. Fitzpatrick et al. (2000b) assumed that acceleration and deceleration occur at outside the limits of curve and they are both equal to 0.85 m/sec^2. Ottesen and Krammes (2000) found that the deceleration and acceleration before and after a curve were assumed to take place on the tangent only at a rate of 0.85 m/sec^2.

From the study Cafiso et al. (2005) it is found that the entry steering manoeuvre usually starts 50 m to 70 m before the beginning of the curve with a progressive increase of curvature that continues inside the curve reaching a minimum value approximately equal to the inverse of the curve radius. The steering manoeuvre to come out of the curve starts inside the curve and lasts until the straight trajectory at the beginning of the successive tangent is reached.

It can be observed that with the exception of radius of curve or degree of curve, which are reflected in majority of models, no uniformity exists among the operating speed models in terms of predictor variables. The number of curves used for data collection also varies a lot. These discrepancies may be attributed to the fact that these models were developed in different countries, where road and traffic conditions vary. Also it is expected that the driver decelerates the vehicle approaching the middle of the curve and accelerates after this point.

2.5.4.3 Operating speed models for vertical curves

Vertical alignment determination is a major element of highway design that has important implications on road construction costs, traffic operations, vehicle fuel consumption, driving manoeuvrability and safety (Jessen et al. 2001). As such there are not many models available in literature to measure design consistency of vertical curves. Vertical curves on horizontal tangents were divided into three categories: non-limited sight distance (NLSD) crest curves, limited sight distance (LSD) crest curves, and sag curves. Sight distance is the primary factor that affects driver expectancy when looking at the vertical alignment of the road (Messer 1980). Proper crest curvature design is critical for safe vehicle operation because the roadway effectively blocks the view of the driver approaching a crest of vertical curve. On account of this accident rates are higher at crest than those on level tangent sections of roadway (Jessen et al. 2001).

Fambro et al. (1999) studied the relationship between operating speed and design speed at crest vertical curves. Their study of 42 curves in three states also concluded that "the inferred design speed of a crest curve (without shoulders) is a moderately good predictor of operating speeds for these types of roadways". They developed an operating speed model for two lane rural roads with shoulders less than 1.8 m wide and determined that operating speeds are well above the inferred design speeds on crest vertical curves. However, a statically significant relationship between inferred design speed and operating speed could not be established for highways with shoulder width greater than 1.8 m. The developed model is:

$$V_{85}=72.74+ 0.47/ V_d \qquad \text{-- (2.37)}$$

Fitzpatrick et al. (2000a) found operating speeds on crest vertical curves with LSD and sag vertical curve are a function of the rate of vertical curvature (K). For vertical crest curve with NLSD (i.e., K> 43m/%), it was recommended to use desired speed.

The developed models are:

V_{85}=111.07 – 175.98/K (Crest, LSD) R^2=0.54 -- (2.38)

V_{85}= 100.09- 126.078/K (Sag) R^2=0.68 -- (2.39)

Based on data collected from 70 sites, Jessen et al. (2001) developed operating speed models for LSD and NLSD vertical curves. In their study posted speed(V_p) and average daily traffic (ADT) are found to be good predictor variables.

V_{85}=86.8+0.29V_p- 0.614G1- 2.39x10^3ADT (LSD) R^2=0.54 -- (2.40)

V_{85}=72.1+0.432V_p -2.12x10^3ADT (NLSD) R^2=0.42 -- (2.41)

Non-limited sight distance (NLSD) for the study was defined as adequate distance for a comfortable stop on wet pavement after perceiving an object on the vehicle path when the initial speed of a passenger car is 77 km/h. The crest curve geometrics would be expected to produce an operating speed consistent with desired speeds along roadway tangent segments. Fitzpatrick et al. (2000) models are based on geometric characteristics of curves while those of Fambro et al. (1999) and Jessen et al. (2001) models reflect traffic characteristics.

2.6 DESIGN CONSISTENCY BASED ON SAFETY CONSIDERATIONS

The ability for drivers to be able to accelerate, decelerate, and change direction is dependent on there being sufficient friction available at the contact surface between the vehicle and the road. The friction available is influenced by numerous factors; one of which is the road surface condition. As the vehicle travels on a horizontal curve, both the vehicle and the passengers are subjected to centrifugal forces acting on the outside of the curve. These forces are balanced by a combination of forces generated by the side (lateral) friction between the road surface and the tyres and a component of the vehicle weight caused by the superelevation (Khanna and Justo 2001). Designers should ensure that the lateral forces are kept within a limiting range that will ensure both driver's comfort and vehicle stability (Nicholson 1998). Safety considerations showed that safe traffic operation and efficient driver performance are usually associated with consistent highway design. The aspects of safety consideration are vehicle stability and low cost improvements (Gibreel et al. 1999).

2.6.1 Vehicle Stability

Vehicle Stability is an important measure of design consistency as it directly influences road safety. The centripetal acceleration is sustained by the side friction between the tyres and pavement and by a component of gravity if the road is superelevated. When a roadway lacks vehicle stability, it means that side friction available is insufficient to withstand excessive centrifugal forces experienced on a vehicle moving on horizontal curve which may lead the vehicle to slide out or lead on to collision. The geometric design will have little effect on the available friction, but it can influence the friction demand (and hence margin of safety) by influencing the behaviour of drivers (and particularly their choice of speed) (Nicholson 1998).

Several researchers have recommended using vehicle stability as a criterion to evaluate design consistency and safety (Lamm et al. 1991, Morrall and Talarico 1994, Nicholson 1998). However, although Lamm et al. (1991) and Morrall and Talarico (1994) based the margin of safety on the difference between side friction supply and side friction demand, Nicholson (1998) based it on the difference between operating speed and safe (limiting) speed.

The difference between side friction available and side friction demanded, which is denoted as Δf_R, is used to represent vehicle stability. Lamm et al. (1991) suggested a quantitative approach to evaluate design consistency based on vehicle stability that was represented by the difference between side friction supply and side friction demand. The values of side friction supply f_R were determined for each curve using table value of AASHTO guide. The values of side friction demand f_{RD} were calculated for each curve using the AASHTO formula, as follows:

$$f_{RD} = \frac{V_{85}^{2}}{127R_c} - e \qquad \text{-- (2.42)}$$

Where,

f_{RD} = Side friction demand and

V_{85} =Observed operating speed (km/h).

R_c = Radius of curve (m)

Using the data collected, different models were developed by Lamm et al. (1991) to relate side friction supply and side friction demand to the observed operating speed, as follows:

$$f_R = 0.082 + 4.692 \text{ X } 10^{-3} V_{85} - 7\text{X}10^{-5} \ V_{85}^{\ 2} \qquad R^2 = 0.74 \qquad - (2.43)$$

And

$$f_{RD} = 0.253 + 2.33 \text{ X } 10^{-3} V_{85} - 9\text{X}10^{-5} \ V_{85}^{\ 2} \qquad R^2 = 0.56 \qquad - (2.44)$$

If side friction demand exceeds side friction supply, concerns for vehicle stability on the curve would arise. The side friction model was developed by Bonneson (1999) based on the hypothesis that drivers will modify their side friction demand based on a desire for both safe and efficient travel.

Side friction demand,

$$f_{RD} = 0.256 - \ 0.0022V_a + B(V_a - V_c) \qquad R^2 = 0.88 \qquad -- (2.45)$$

Where,

$$V_c = 63.5R \ \left(-B + \sqrt{B^2 + \frac{4c}{127R}} \ \right) \leq V_a \qquad -- (2.46)$$

With $c = \frac{e}{100} + 0.256 + (B - 0.\ 0022V_a)$ -- (2.47)

$$B = 0.0133 - 0.0074 I_{TR} \qquad -- (2.48)$$

Where,

e =superelevation rate in per cent

I_{TR} = Indicator variable (=1.0 for turning roadways; 0.0 otherwise).

V_a = 85th percentile approach speed km/h

V_c = 85th percentile speed at curve in km/h

The proposed model predicts a decrease in side friction demand with an increase in approach speed. In addition, the proposed model indicates that side friction demand increases with increasing speed reduction (i.e., V_a - V_c). Bonneson (1999) also compared friction factors obtained from the proposed model with those recommended in the Green Book which indicates that a nominal speed reduction of 5–10 km/h is being imposed on drivers travelling on curves designed with near minimum radii. Based on the study, Lamm et al. (1996) and Cafiso et al. (2005) developed the model to predict side friction assumed, considering utilisation ratio of side friction (% of friction factor utilised out of the maximum permissible side friction factor). The side friction assumed is a fraction of tangential friction (f_T) and is taken as being,

$$f_R = 0.925 \times n \times f_T \qquad - (2.49)$$

Where,

n = utilization ratio of side friction.

Based on international experience, this value varies between n = 40% and n = 50% for rural roads. It explains that 87% to 92%, of friction in the tangential direction is still available (f_T /f_{Tmax}) when riding through curves, for acceleration, deceleration, braking, or evasive manoeuvres:

And, $$f_T = 0.59 - 4.85 \times 10^{-3} \times V_d + 1.51 \times 10^{-5} \times V_d^2 \qquad - (2.50)$$

The side friction demanded is expressed as in Eq.2.42

Where,

V_d=Design speed in km/.h

e = superelevation rate (%/100)

The safety margin was defined by Nicholson (1998) as the difference between the limiting speed, V_L and the design speed, V_d. The limiting speed is the speed at which f = f $_{max}$ and e = design superelevation. For circular curve with radius R and design superelevation, e,

V_L is given by, $$V_L = \sqrt{gR(e + f_{max})} \qquad - (2.51)$$

Where, V_L is in km/h and R in m.

Clearly V_L is greater than or equal to the design speed (V_L =V when f = f_{max}). Since the value of f_{max} used in design is generally lower than actual available friction, it represents a lower bound of the safety margin. The superelevation and side friction vary linearly with the degree of curvature. It was concluded that having a margin of safety on a highway section is not enough to achieve a consistent design, but this margin should be consistent. The consistency of the safety margin can be achieved when its standard deviation is small (Nicholson 1998). Based on accident experience, Lamm et al. (1996) suggested evaluation criteria (termed as Safety criteria III), using the difference between f_R and f_{RD} to evaluate design consistency as:

- Good design: f_R - $f_{RD} \geqq -0.01$ (no improvements are required).
- Fair design: $-0.04 \leq f_R - f_{RD} < 0.01$ (superelevation rate must be related to operating speed to ensure that side friction supply will accommodate side friction demand).
- Poor design: $f_R - f_{RD} < -0.04$ (redesign is recommended).

Where, $$f_R = \frac{V_d^{\ 2}}{127R} - e \qquad \text{-- (2.52)}$$

$$f_{RD} = \frac{V_{85}^{\ 2}}{127R} - e \qquad \text{- (2.53)}$$

Easa (2003) developed optimization models which, while maximizing highway design consistency based on the safety margin, eliminate trial and error to assess all AASHTO methods to determine the design superevaluation for a given set of highway curves. Finally, it was noted that side friction is not as easy to recognize and measure as operating speed. Therefore, using a side friction model developed from a specific database along with evaluation criteria from another database may not lead to accurate evaluations. Furthermore, a number of criticisms were made on the applications of vehicle stability as consistency evaluation (Gibreel et al. 1999, Hassen et al. 2001). In vehicle stability, the vehicle is represented by a point mass, rather than a body, which ignores the distribution of frictional forces between different tyres. Besides, the assumption that the vehicle moves with a constant speed and that drivers follow a path with a certain radius equal to the curve radius is invalid (Gibreel el al. 1999). Also, it was concluded from the study that margin of safety on a highway section is not enough to achieve a consistent design (Nicholson 1998). Safety criterion III gives useful information when it is combined with other two criteria (Safety criterion I and Safety criterion II). Because of all these reasons Cafiso et al. (2005) suggested to use safety consideration as a supplement method design consistency measure with other methods of design consistency measures.

2.7 DESIGN CONSISTENCY BASED ON PERFORMANCE CONSIDERATIONS

A consistent highway design can be achieved with the driver's performance, which leads to decrease of driver's error and produces safe operation (Gibreel et al. 1999). Driver workload is a measure that has been used in the measurement of design consistency. Driver workload is defined as the time rate at which drivers must perform a given amount of work or driving tasks (Messer 1980). The mental work load is important as the driving task involves information processing and decision making. By measuring the amount of incoming information to the driver, a measure of the workload imposed on the driver can be obtained (Fitzpatrick 2000a). The mental workload increases with increases in the complexity of the driving situation (Messer 1980) and increases with decreases in the time available for processing information and making decision (Nicholson 1998).

Highway designers should avoid highway sections that have very low or very high driver workload. The first condition may cause drivers to become bored or tired whereas the latter condition may confuse drivers and cause them to misinterpret an unexpected development or respond inappropriately (Hassen et al. 2001). The mental workload increases with reduction in sight distance (Messer 1980). Hence for safety reasons, the horizontal alignment must be consistent in terms of sight distances. Visual demand is considered to be a surrogate for driver workload because driving is essentially visual in nature. Visual demand is defined as the amount of visual information needed by the driver to maintain an acceptable path on the roadway (Wooldridge et al. 2000a). Large differences and abrupt changes in horizontal alignment should be avoided so that driver workload is not excessive. Significant changes in driver workload requirements often lead to crashes. By limiting the workload imposed on the driver to acceptable levels, the likelihood of overloading the driver's mental capacity is reduced (Wooldridge et al. 2003).

Messer (1980) developed a regression model to estimate the expected workload value on a specific feature on a road's section

$$WLn = (UxExSxRf) + (C + WLi) \qquad \text{-- (2.54)}$$

Where,

WLn = Expected workload value for the specific features

U= driver unfamiliarity factor (depends on highway classification and location)

E= Feature expectancy factor

S= Sight distance factor

Rf = Average workload potential value for the general feature.

C = Carry factor (depends on separation distance between features) and

WLi = Workload value of the previous section

Messer (1980) suggested a rating scale from (1 to 6) to estimate an average workload value Rf on general individual geometric features. He classified the design consistency in a range extending from (WLn<1): consistent design (no problem expected) to WLn > 6: Inconsistent (big problem possible).The driver workload procedure quantifies design consistency by computing a value for driver workload. The good scales include the Cooper-Harper Scale, Subjective Workload Assessment Technique (SWAT) and the NASA Task Load Index (TLX). SWAT and TLX are multi-dimensional and, thus, are more diagnostic than the Cooper-Harper Scale; however, SWAT and TLX are rather cumbersome and require a great deal of driver training to be an effective tool for estimating workload. The Cooper-Harper scale is a 10-point Likert scale anchored by adjectives at each level. It can easily be taught to drivers (Wierwille and Eggemeier 1993(cited in Fitzpatrick 2000b)).

Wierwille and Eggemeier in 1993 also identified the measures like heart rate (HR), heart rate variability (HRV), brain activity, and eye activity to assess the performance of a driver (Fitzpatrick 2000b). Heart rate changes with workload have long been documented as a general index of arousal and/or physical work. Changes in heart rate variability, i.e., the variance in the beat-to-beat interval (cardiac arrhythmia) have been found to reliably discriminate among various types of tasks relevant to driving. As workload increases, HRV decreases, while HR tends to increase with physical effort. Brain activity measurement presents a much more complex recording and data interpretation situation. Eye activity measurements have been in use for many decades. Eye blinks and eye movements have all been studied for application to workload assessment. Eye blink frequency and duration have both been shown to have a direct relationship with visual workload: blink rate decreases as workload increases, as does blink duration. Under stress, drivers tend to blink less, stare more, and minimize time that their eyes are closed during each blink. But, eye blink measurements do not correlate well with other types of cognitive tasks or those involving other sensory modalities.

Krammes et al. (1995b) examined design consistency for horizontal curves using vision occlusion to study driver's workload. Drivers were asked to voluntarily shut their vision off and to open their eyes only when they think it is necessary to extract information from the highway for guidance. The amount of time that the drivers were unwilling to have their vision shut off over a fixed length of the roadway represented the mental workload required for the guidance task. The workload profile for a highway segment was established using the average workload on the tangent and the average workload on the curve and concluded that the locations of higher workload values would have a relatively high accident rate that probably resulted from a poor design.
The changes in road geometry was also studied by Fitzpatrick et al. (2000b) using vision occlusion method, who concluded that driver workload increase with the inverse of radius and effect of deflection angle was persistent but small in overall influence. Using the visual occlusion technique, Wooldridge et al. (2000a, b) measured a driver's visual demand on horizontal curves as a percentage of time a driver observes the roadway. They studied the effects of variations of curve radius, deflection angle, spacing, and sequences revealed several relationships between roadway geometry and visual demand. Curve radius and its reciprocal were found to be significantly related to visual demand of both familiar and unfamiliar drivers. In spite of its importance in evaluating design consistency; analytical models are not readily available to quantify driver workload.

It was concluded, however, that the use of driver workload as a measure of consistency was much more limited than operating speed. The principal weakness of the driver workload concept is that it is difficult to measure such subjective estimates and, hence, to validate such models (Gibreel et al. 1999, Fitzpatrick et al. 2000b).

2.8 DESIGN CONSISTENCY BASED ON ALIGNMENT INDICES

Alignment indices are quantitative measures of the general character of a roadway segment's alignment. They are the function of the dimensions of horizontal and/or vertical alignment elements. Therefore, they will provide a mechanism for quantitative assessment of successive elements from a system-wide perspective, which is a fundamental motivation of design consistency research (Fitzpatrick et al. 2000b). Anderson et al. (1999) reported a number of advantages in using alignment indices: they are easy for designers to use and understand, they can provide a mechanism for quantitatively comparing successive geometric elements from a system-wide perspective, and they attempt to quantify the interaction between the horizontal and vertical alignments. Geometric inconsistencies are measured when there is a large increase/decrease in the values of alignment indices for successive roadway segments or a high rate of change in alignment indices over some length of roadway or a large difference between the individual feature and the average value of the alignment index. Alignment indices hold some promise for design practice in the United States. However, the different design characteristics and use of these indices must be considered, when attempting to apply the alignment indices in the United States and Germany (Fitzpatrick et al. 2000b).

Polus (1980) investigated the relationship between longitudinal geometric measures (such as the average radius, or the ratio between the minimum and maximum radius of an alignment) and safety levels on two lane rural highways. Polus concluded that safety correlates with a similarity in design elements and, therefore, with consistency. Polus also observed in his study that drivers tended to build up an expectation of what the upcoming roadway would be like, based on their immediate previous driving experience.

Polus and Dagan (1987) suggested several horizontal alignment indices which include the average curvature in degrees per kilometre, the average radius of curvature, the ratio of the maximum to minimum radius, and the ratio of the average radius of curve to minimum radius. They evaluated the correlation between these indices and accident rates and found that only the ratio of the maximum to the minimum radius was significantly related to the accident rate. They also observed that as these value approaches 1, or as the radii of the roads become more consistent, a reduced accident rate may be expected.

Anderson et al. (1999) have stated that the ratio of maximum to minimum radius is not recommended as a design consistency measure due to its relatively low sensitivity to collision frequency compared to the other alignment indices AR (average radius), AVC (average rate of vertical curvature), and CRR (R_c/AR) studied. They found that safety is sensitive to CRR, and subsequently suggested using this ratio as a consistency measure. The reasoning behind selecting this measure was given as, when the radius of a horizontal curve deviates greatly from the average radius along the roadway section, that curve may violate driver expectancy, creating inconsistency (Anderson et al. 1999). Table 2.2 lists the alignment indices considered for the speed prediction and design consistency by Fitzpatrick et al. 2000a, b).

Ratio of an individual curve radius to the average radius, average rate of vertical curvature, and average radius of horizontal curvature are the alignment indices which explain the consistency of roadway section, but are not as strongly related to safety as speed reduction (Fitzpatrick et al. 2000c). Alignment indices did not explain the variation in measured speeds on long tangent. It may be an appropriate measure to supplement speed reduction in a design consistency methodology. Therefore, they should not be considered as the primary measures in design consistency evaluation (Fitzpatrick et al. 2000c).

Using alignment indices, however, still lacks an explicit criterion for consistency evaluation and has not been strongly correlated to safety yet (Hassan et al. 2001). However, there are some rules pertaining to geometric design features such as degree of curve (DC) and curvature change rate of a single circular curve with transition curve (CCR) under safety criterion of successive elements (Lamm et al. 1996), which are included in some European design standards as in Table 2.3.

Table 2.2 Alignment Indices Selected for Speed Prediction Evaluation

Horizontal Alignment Indices	**Vertical Alignment Indices**
• Curvature Change Rate - CCR (deg/km) $= \frac{\sum \Delta i}{\sum Li}$ Where, Δ = deflection angle (deg) L = length of section (km)	• Vertical CCR - V CCR (deg/km) $= \frac{\sum Ai}{\sum Li}$ Where, A = absolute difference in grades (deg) L = length of section (km)
• Degree of Curvature - DC (deg/km) $= \frac{\sum DCi}{\sum Li}$ Where, DC = degree of curvature (deg) L = length of section (km)	• Average Rate of Vertical Curvature - V AVG K (km/percent) $= \frac{\sum \frac{Li}{\lvert Ai \rvert}}{n}$ Where, L = length of section (km) A = algebraic difference in grades (%) n = number of vertical curves
• Curve Length: Roadway Length - CL:RL $= \Sigma(CL)i/_{\Sigma Li}$ Where, CL = curve length (m) L = length of section (m)	• Average Gradient - V AVG G (m/km) $= \Sigma\Delta Ei/_{\Sigma Li}$ Where, ΔE = change in elevation between VPI_{i-1} and VPI_i (m) L = length of section (km)
• Average Radius - AVG R (m) $= \Sigma Ri/_{n}$ Where, R = radius of curve (m) n = number of curves within section • Average Tangent - AVG T (m) $= \Sigma(TL)i/_{n}$ Where, TL = tangent length (m) n = number of tangents within section	**Composite Alignment Indices** • Combination CCR – COMBO (deg/km) $= \frac{\Sigma\Delta}{\Sigma Li} + \frac{\Sigma Ai}{\Sigma Li}$ Where, Δ = deflection angle (deg) A = absolute difference in grades (deg)

Table 2.3 Design consistency evaluation criteria based on Alignment indices

Good design level:	Fair design level:	Poor design level:
1.(DC_i – DC_{i+1}) ≤ 5º/100ft 2.(CCR_i–CCR_{i+1}) ≤ 180 gon/km	1. 5º/100ft <(DC_i–DC_{i+1}) ≤ 10º/100ft 2. 180 gon/km < (CCR_i–CCR_{i+1}) ≤ 360 gon/km	1.(DC_i–DC_{i+1})> 10º/100ft 2.(CCR_i–CCR_{i+1}) > 360 gon/km

Where,

CCR = curvature change rate in gon/km

DC = Degree of curvature (deg/100ft)

Conversion factor:

DC (deg/100ft) X 36.5= CCR (gon/km) valid only for circular curve

Following are the conclusions drawn from literature study on alignment indices:

- Alignment indices are the direct measures of consistency. They are easy to calculate and explain.
- Alignment indices explain the general character of an alignment rather than individual section.
- The alignment indices should be used as a consistency measure to supplement along with other methods of design consistency.

2.9 DESIGN CONSISTENCY BASED ON SPEED DISTRIBUTION MEASURES

Drivers travel at desired operating speed, which is the speed at which they would operate if unimpeded by other traffic. Desired speeds depend on roadway condition, weather, environment, and roadway geometry; thus, they cannot be measured directly. As a result, a common assumption made is that desired speeds are directly related to free flow speeds and can be approximated using a sample of free flow vehicle speeds. A common hypothesis in traffic flow theory is that speeds, particularly of free-flowing vehicles, are normally distributed. It has been concluded that vehicle speeds on roadways follow a continuous distribution and that distribution measures could identify geometric deficiencies (Collins et al. 1999, Fitzpatrick et al. 2000b). Thus, free flow speeds and the statistical measures associated with them may identify alignment deficiencies.

Alignment features exhibiting higher values of speed variability have been identified as potential locations for driver's error. Significant changes in speed distributions may suggest that design inconsistencies are present at that alignment feature. Speed distribution measures include mean speed, variance, standard deviation, coefficient of variation, skewness, and kurtosis that are logical candidates for a consistency rating method (Fitzpatrick et al. 2000b).

Research has been carried out to develop the relationship between speed distribution measures and roadway characteristics that may identify inconsistent roadway features. Traditionally, higher speed variability suggests a higher accident potential. The research results support this statement and relate speed variance to accident rates. It was confirmed from the study that a direct relationship exists between speed variance and accident potential (Garber and Gadiraju, 1989, Collins et al. 1999, Fitzpatrick et al. 2000b). Accident rates increase with increasing speed variance for all classes of roads and the accident rate on a highway does not necessarily increase with an increase in average speed (Garber and Gadiraju, 1989). In a study Garber and Gadiraju (1989) defined a relationship for variance and average speed as follows:

$$s^2 = 16.7 - \frac{204803}{V_M^2} \qquad \text{- (2.55)}$$

Where,

s^2 = speed variance in $(mph)^2$

V_M = mean speed, $25 < V_M < 70$ mph

The results of the analysis also suggested that a relationship exists between speed variance and design speed. Also, the difference between the design speed and the posted speed limit has a significant effect on the speed variance. The analysis included a model that related speed variance and design and posted speeds as follows:

$$s^2 = 57 + 0.05(V_D - V_P - 10)^2 \qquad \text{-- (2.56)}$$

Where,

V_D = Design speed in mph

V_P = Posted speed limit in mph

Speed variance on a highway segment tends to be at a minimum when the difference between design and posted speed is between 8 and 16 km/h (Garber and Gadiraju, 1989). A positive correlation exists between the coefficient of variation on a horizontal curve (R_c> 100 m) and that on its preceding tangent (Collins et al. 1999). The mean vehicle speed and 85th percentile speed on the horizontal curve are 2.8 km/h and 3.0 km/h lower on the curve than on the tangent. In addition, the speed variance was 9.5 percent lower on the curve than on the preceding tangent. But, in general, a low correlation was observed between geometric features and speed variance (Collins et al. 1999).

Fitzpatrick et al. (2000b) tried speed variance as a design consistency measure assuming that speed variance increases at locations with potentially inconsistent designs such as sharp horizontal curves. But the speed variance decreased on horizontal curves as compared with the upstream tangent. This finding is consistent with the observation that horizontal curves affect the speeds of faster vehicles more than that of slower vehicles, thus reducing the speed variance. Hence it was concluded that increase in speed variance may be an indicator of potential safety problems for some geometric design features or traffic situations, but it is not useful in explaining safety difference between tangents and horizontal curve on two-lane rural highways (Collins et al. 1999, Fitzpatrick et al. 2000b).

In another study Partha et al. (2008) found that the speed of an individual vehicle type is normally distributed and the speed data of all vehicles combined at a section of highway may or may not follow a normal distribution. So the parameter Spread Ratio (SR) defined as the ratio of the difference between 85th and 50th percentile to the difference between 50th and 15th percentile speed was introduced to explain the variation in the speed of different categories of vehicles. The speed data are normally distributed only when SR is in the range of 0.69 – 1.35.

2.10 RELATIONSHIP BETWEEN SAFETY AND CONSISTENCY MEASURES

Traffic collisions are a major source of loss of life and contribute to a number of serious to minor injuries (grievous to simple injuries). Since most drivers travel as fast as they feel comfortable and slow down only where necessary, rural highways with lower design speeds exhibit uneven operating speed profiles (Krammes 1997), i.e. drivers accelerate to their desired speed on tangents and gentle curves and decelerate only on sharper curves. In some situations, some of the drivers do not reduce their speed as much as required, which summarises accidents on curves.

Crash studies have shown that the larger the speed reduction required from the preceding tangent to the subsequent curve, the higher the crash rate on the curve. Baldwin in 1946 found that while the accident rate increased as the radius of horizontal curves decreased, the accident rate for small radius curves generally decreased as the frequency of curves (per length of highway) increased (cited in Nicholson 1998). Babkov (1975) reported that the lack of proper sight distance was the main reason for about 8% -10% of the accidents in the former Soviet Union. Because of the influence of a centrifugal force and restricted visibility about 30 percent of all accidents on rural roads occur at bends (Stewart and Chudworth 1990).

The accident rate on horizontal curves was studied by Brenac in 1996 who concluded that the probabilities of accident occurrence on two-lane rural highways are especially high at horizontal curve. In addition, it was confirmed that accident rate decreases with the increase of the radius of horizontal curve (cited in Gibreel et al. 1999). Landge et al. (2000) classified the various types of variables which were used to develop road safety models as traffic variables (ADT, speed, non-motorised/pedestrian traffic and traffic composition), road geometric variables (lane width, shoulder width and its type, number of lanes, medians etc.), accident variables, and other variables. Sikdar (2005) used GIS database to investigate hazardous road sections in Raigad district of Maharashtra and found that curve radius <350 m and narrow bridges (relative bridge width <1) contributes to high rate of fatal accidents.

Vivian (2006) found that roadway width, traffic volume, percentage of heavy vehicles, and pedestrian traffic are the factors affecting the prediction of accidents. In a study on geometric design elements and safety of Australian rural roads it was found that crash rate increases with decrease of radii below 400 m and increase of gradient and reduces with increase of lane width up to 3.6 m (Veith and Turner, 2006). Chandra and Prasanta (2004) analysed ten years' accident data of different states' two lane roads in India and found that 1.05 percent of road accidents occur due to bad condition of the road. To predict accident models the section of a road and its shoulder were ranked on a 5 point scale as Excellent – 5, Good – 4, Average – 3, Poor – 2, and Bad – 1. The developed accident prediction model showed that number of accidents per kilometre – year increases with AADT and decreases with improvement in road/shoulder condition.

To quantify the speed variation, Fitzpatrick et al. **(**2000a**)** studied 5,287 individual horizontal curves in Washington and classified it as good, fair, and poor with respect to design safety using the accident frequencies, exposure vehicle kilometre of travel, and accident rates. The developed criteria are as given in Table 2.4.

Table 2.4 Accident Rates at Horizontal Curves by Design Safety Level

Fitzpatrick et al. **(**2000a**)**

Accident/Million veh.km	Exposure (million veh-km)	Design safety levels
0.46	3,206.06	Good: $\Delta V_{85}= V_{85i} - V_{85i+1} < 10$ km/h
1.44	150.46	Fair: 10 km/h< ΔV_{85}<20 km/h
2.76	17.05	Poor : $V_{85} > 20$ km/h

In their study the following statistically significant Poisson regression model was obtained.

$$Y = \exp(-7.1977)\, AADT^{0.9224}\, CL^{0.8419} \exp(0.0662\, SR) \qquad \text{-- (2.57)}$$

Where,

Y=Number of accidents that occurred on the curve during a three-year period.

AADT= Annual average daily traffic volume (Veh/day)

CL=Horizontal curve length (km)

SR=Speed reduction on horizontal curve from adjacent tangent or curve (km/h)

Also, the accident frequency analysis was supported to use the speed reduction as a design consistency measure. In a study Anderson et al. (1999) used alignment indices as design consistency measures for roadway. In their study accident frequencies were modelled as a function of exposure (AADT and section length) with each of four alignment indices - average radius, ratio of maximum radius to minimum radius, average tangent length, and average rate of vertical curvature. The alignment index ratio of maximum radius to minimum radius (RR) for the road section had a statistically significant relationship to accidents, but failed to predict accident frequency. In their study, ratio of individual curve radius to average curve radius (CRR) was found quite sensitive to the accident frequency and the following statistically significant Poission regression model was obtained:

$$Y=\exp(-5.932)\, AADT^{0.8265}\, CL^{0.7727} \exp(-0.3873CRR) \qquad \text{-- (2.58)}$$

Of the candidate design consistency measures Fitzpatrick et al. (2000a, b and c) identified that predicted speed reduction by motorists on a horizontal curve relative to the: 1) preceding curve or tangent, 2) ratio of an individual curve radius to the average radius for the roadway section, 3) average rate of vertical curvature for a roadway section, and 4) average radius of curvature for a roadway section. Anderson and Krammes (2000) estimated the reduction in 85th percentile speeds from the approach tangent to the midpoint of the following curve. They found that a statistically significant relationship existed between mean speed reduction and mean accident rate: sites with higher speed reductions showed higher accident rates. All these important findings were further investigated in this research through the development of a relationship between alignment indices and accident frequency, as a measure of the design consistency of intermediate lane rural highways.

2.11 FINDINGS

This literature review compiles different measures for evaluating geometric design consistency, as presented in available research work, to identify their potential applicability and any required future research. The consistency measures, i.e. operating speed, vehicle stability, alignment indices, speed variance, and driver workload are presented. Emphasis is placed on the studies on consistency evaluation estimated for passenger cars.

From the findings of this review of design and operating speeds, it can be summarised that, because of drawbacks in design speed concept, design consistency based on operating speed concept is the common method available. As for horizontal alignment alone, minimising differences of operating speeds between successive roadway elements is the best available method to achieve consistency of alignment and cross-section. The 85^{th} percentile of a sample of speeds measured at a specific location is generally accepted as a measure of the operating speeds at that location. Therefore, the ability to predict the 85^{th} percentile speed using geometric variables is critical to the operating speed based methods. All these design consistency methodologies are based on assumption that speed distribution on successive elements is the same. Actually, the speed distributions on successive elements are different and each driver responds differently to the horizontal curve based on his/her desirable tangent speed and the actual side friction factor. Thus, it is preferable to obtain and analyse the speed distribution of individual vehicles between successive elements than speed change by comparing between the 85^{th} percentiles of the two distributions.
The literature review also revealed that it is insufficient to just collect the speeds at a single point on curve of the road. Curve speeds are dependent upon the geometry of curve and/or the approach speeds. Thus, in addition to the speed on the curve, other factors need to be considered. It can be observed that radius of curve(R_c) and/or the degree of curve are reflected as explanatory variables in majority of models; no uniformity exists among the operating speed models in terms of predictor variables. The number of curves used for data collection also varies a lot. These discrepancies may be attributed to the fact that these models were developed in different countries, where road and traffic conditions vary. It can be summarised from observed models that deceleration occurs from tangent to curve and acceleration starts from centre of curve to its end. Also, it is observed that geometrics are the main

influencing variables in the prediction of operating speed at horizontal and vertical curves.

Vehicle stability is another important issue in ensuring safe traffic operation. Different criteria to evaluate design consistency based on vehicle stability have been developed in terms of the difference between side friction supply and demand or the difference between operating and safe (limiting) speeds. However, these criteria are based on a design formula that has been the subject of considerable criticism. Also, it is very difficult to develop accurate and comprehensive models for side friction supply and demand. Therefore, this method is not considered further for consistency evaluation of intermediate lane rural highways.

Driver workload is a measure of the information processing demands imposed by roadway geometry on a driver. Vision occlusion, subjective difficulty ratings, and eye-mark system were the driver workload procedure quantifies design consistency by computing a value for driver workload. The locations of high driver workload or those that do not coincide with the driver's anticipation usually show poor design. It was concluded in the literature that the use of driver workload as a measure of consistency was much more limited than operating speed. Also, it is difficult to measure and estimate driver's workload.

Alignment indices are quantitative measures of the general character of a roadway segment's alignment. They have been used in other countries, specifically Germany, England, and in the United States, as a measure of the design consistency of their roads. Proposed indicators of geometric inconsistency occur when there is a large increase or decrease in the magnitude of the alignment indices for a successive roadway segment or feature or when a high rate of change occurs over some length of road. They are easy for designers to use, understand, and explain. Alignment indices are used as a supplementary consistency measure along with speed based method of design consistency evaluation.

It was also concluded in the literature that speed variance is inappropriate to consider as a design consistency measure between successive features on two lane highways.

Hence this method is not considered further for consistency evaluation of intermediate lane rural highways.

A relationship between design consistency measures and safety, which is the main objective of the design consistency concept, is also discussed. At present lack knowledge of how best to coordinate alignment and cross-section to predict accidents on intermediate lane rural highways.

2.12 SUMMARY

An overview of various approaches available on geometric design consistency has been presented in this chapter. The design consistency approaches such as, speed considerations, safety considerations; performance considerations, alignment indices and speed variance have been briefly discussed. The available operating speed and speed differential models for horizontal curves and available operating speed models for vertical curves have also been indicated. Besides the discussion on relationship between safety and design consistency and findings of the literature review were also indicated.

CHAPTER 3

CONSISTENCY EVALUATION OF HORIZONTAL CURVES

3.1 INTRODUCTION

Achieving highway geometric design consistency is an important issue in the design and evaluation of rural highways to attain smooth and safe traffic operation. To develop design guidelines for the geometric design consistency on intermediate lane rural highways, it is necessary to collect the required data. A brief discussion of the strategy adopted for the collection of required data is presented in this Chapter along with the details of the study sites and the methodology adopted for the retrieval of data. Efforts made to develop the operating speed and speed differential models including their validation for simple horizontal curves are also discussed. This Chapter also presents the different design consistency evaluation criteria considered for horizontal curves.

3.2 DATA REQUIREMENTS

On the basis of the literature review (Lamm et al.1988, Gibreel et al. 1999, Fitzpatrick et al. 2000a, Mawjoud and Sofia 2008), to develop the highway design consistency evaluation methodology, it was decided to adopt a speed consideration approach. Therefore, the basic data requirements for the consistency evaluation of road stretch are: 1) Geometrics 2) Speed and 3) Accident data. The geometric data include the details such as radius and length of the curve, degree of curvature, deflection angle, super elevation, gradients, sight distance, road width, and shoulder width. The speed data include speed of passenger vehicles at different locations of each site. Accident data include the type and location of accident.

3.3 STUDY AREA

Dakshina Kannada (D.K.) is one of the most commercially and historically important districts in the state of Karnataka,India. The district is well-connected to other places by road, rail, air and water. This district has a total area of 4866 km^2 of land and 60 km of seashore length. Several multinational companies, MRPL, MCF, ONGC, etc are investing in the district. New Mangalore port also contributes to high commercial activities in the district.

The population of Dakshina Kannada district increased from 16,56,165 in 1991 to 18,97,730 in 2001 with a record growth of 14.5 per cent. According to the 2011 census, Dakshina Kannada has a population of 20,83,625 roughly equal to the nation of Macedonia or the US state of New Mexico. Its population growth rate over the decade 2001-2011 was 9.8 %. The district has a population density of 457 inhabitants per square km. This gives it a ranking of 220th in India (out of a total of 640) (http://en.wikipedia.org/wiki/Dakshina_Kannada).

The district has the highest density of vehicles in Karnataka state which is only second to the capital city, Bangalore. Karnataka has earned the distinction of being the state with the third highest accident rates in India, after Tamil Nadu and Maharashtra. Statistics with the National Crime Records Bureau indicate that at least 23 people die every day in Karnataka out of which five are in Bangalore (National Crime Records Bureau- timesofindia.indiatimes.com).

The growth of motor vehicles in Dakshina Kannada district is phenomenal. The fact that the number of motor vehicles increased from 2, 13, 503 in 2003 to 4, 29, 543 in 2011 shows two-fold increase over eight years. Almost all the roads in the district are historical routes widened as vehicle population increased, and its alignment does not conform to any rule that can ensure design consistency. This condition is common for many intermediate lane rural roads as well.

Due to increase in population as well as number of vehicles in the district, there is an alarming rise in number of accidents, as shown in the Table 3.1. Therefore, it is necessary to consider road safety measures to reduce the number of crash cases.

Table 3.1 Accident statistics of Dakshina Kannada district (Source: Office of the S.P, D.K. district)

Year	Road Accidents	Persons Injured	Persons killed
2003	1740	2433	242
2004	1891	2592	235
2005	2028	2709	235
2006	1869	2613	240
2007	1913	2725	240
2008	1895	2601	258
2009	1867	2552	261
2010	1796	2957	280

3.3.1 ROADS SELECTED FOR STUDY

The study is limited to rural highways with carriageway width of 5.5 m. The roads selected for study are listed in Table 3.2 and the types of roads passing through the district are shown in Fig.3.1.

Table 3.2 Road stretches selected for in Dakshina Kannada district

Sl.No	Road	Length (km)	Start	End
1	SH-37	82.4	Subrahmanya	Belthangady
2	SH-64	89.33	Charmadi	B.C. Road
3	SH-67	23.2	Bajpe	Mulki
4	SH-70	34.3	Belthangady	Mulki
5	SH-88	71.6	Sampaje	Mani
6	SH-101	63.9	Suratkal	Kabaka
7	SH-114	16	Kulkunda	Gundya
8	NH-13	30	Nanthoor	Moodabidri

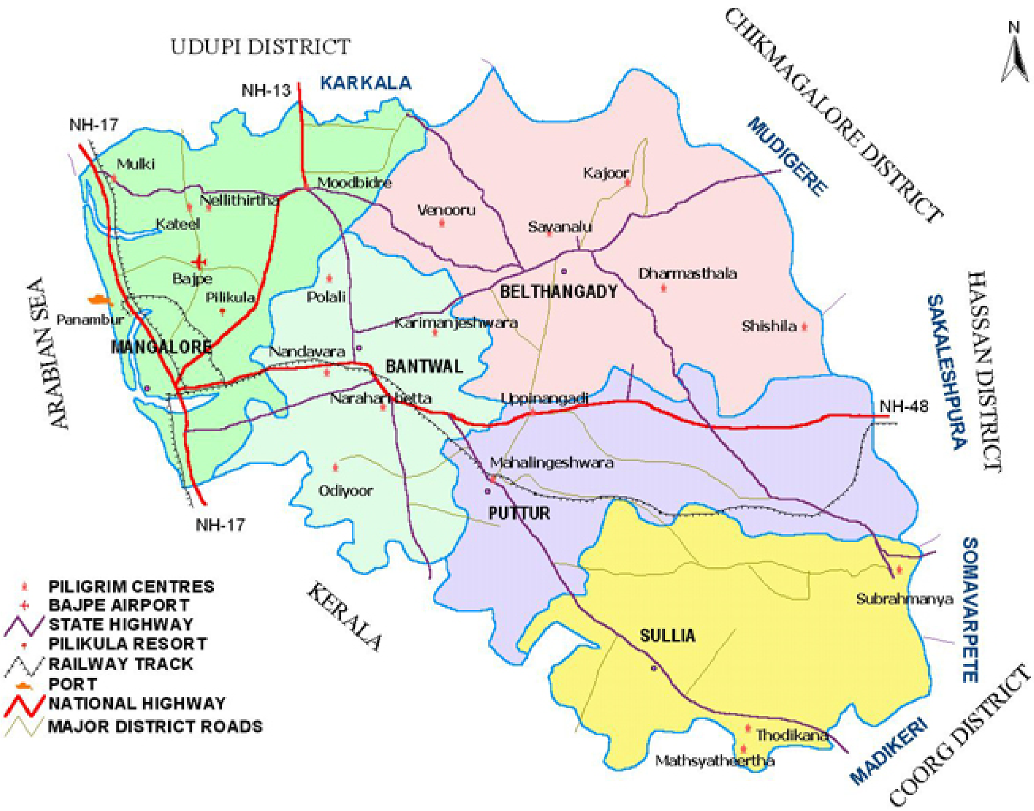


Fig 3.1 Road map of Dakshina Kannada district

(www.dk.nic.in/dtmap.htm)

3.3.2 SITE SELECTION

The selected intermediate lane rural highway sites had good pavement conditions. A site consisted of two directions of travel on horizontal curves and their approach tangent. A number of simple horizontal curves were selected for study. The site that satisfies the following criteria was selected for study, so that roadside conditions might not have adversely affected the operating speed of vehicles travelling on the curves.

- The sites were not close to towns or developed areas.
- Free from influence of intersections and other adjacent sections.
- Free from physical features such as narrow bridge, schools, factories, or recreational parks or activities adjacent to, or in the course of, the roadway that may create an abnormal changes.
- Carriageway is not marked and shoulders are unpaved.

- Grade of horizontal curves is in between + 2% to – 2%.
- Away from township or developed areas that may significantly affect the speed pattern on the curve.
- Horizontal curve with a minimum 100 m tangent length was considered assuming that it allows the driver to accelerate, reach, and maintain desired speed. Desired speed is the speed of a driver at which they would operate if unimpeded by other traffic (Fitzpatrick et al. 2000b).

3.3.3 STUDY STRETCHES

The topography of the project area ranges from plain to rolling terrain. A reconnaissance survey was conducted to identify the site which satisfies the selection criteria. The data collection goal was to collect data for a minimum of 175 sites. A total of 178 horizontal curves with their approaching tangent length were considered. The stretches and their characteristics considered for study are tabulated in Table 3.3.

Table 3.3 Study sites considered in the selected Stretches

Road	Number of Curves selected for study	From	To
SH -37	26	Subrahmanya	Belthangady
SH -64	25	Charmadi	B.C. Road
SH -67	8	Permude	Maradka
SH -70	47	Belthangady	Mulki
SH- 88	43	Sampaje	Puttur
SH -101	9	Polali	Bajpe
SH-114	13	Kulkunda	Gundya
NH- 13	7	Nantoor	Yadapadavu
Total	178		

3.4 GEOMETRIC DATA

Several sources of geometric data such as PWD records and Google maps were explored for obtaining the necessary data. To obtain more information about geometric details of the identified stretches during field visit, theodolite (Venier) surveying was conducted. The collected details were used to prepare CAD (AUTO CAD 2008) drawings. Using these CAD drawings, the required geometric data of all the curves were retrieved.

From the literature review, it was found that the geometric data required for this study include information about the horizontal curves and the tangents preceding these curves. For a horizontal curve, the data required are radius of the curve (R_c), length of the curve (L_H), deflection angle (Δ), preceding tangent length (PTL), superelevation (e), sight distance (SD), lane and shoulder widths at start, middle and end of the curve. Of these, superelevation, sight distance (SD), Preceding tangent length before speed observation point (PTLS), lane and shoulder widths at start (S), middle (M) and end (E) of the curve were obtained by direct observation at site. The prepared CAD drawings were crosschecked with field measurements to ensure correctness of data taken from these drawings. The details such as radius of the curve (R_c), length of the curve (L_H), deflection angle (Δ) and Preceding Tangent Length (PTL) of curve were obtained from CAD drawings. The geometric data collected are tabulated in Appendix 1, Table A 1.1.

3.4.1. Radius and Deflection Angle

The sharpness of the curve is designated by its radius. The external deflection angle to any point on the curve is the angle between the back tangent and forward tangent. Radius (R_c) and external deflection angle (Δ) of curve are obtained from CAD drawing. The radius of each curve is determined using three-point technique. The external deflection angle is measured as shown in Figure 3.2.

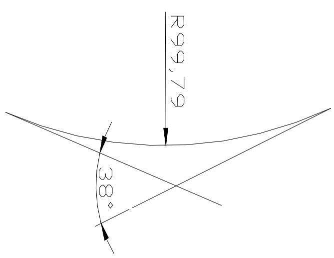


Fig.3.2 Deflection angle

3.4.2 Length of Curve (L_H)

It is the total length of the curve from point of curve to point of tangent. The length of curve is measured from CAD drawing of plan of site.

3.4.3 Tangent Length

Speed on the tangent of the horizontal curve was found to be affected mainly by the tangent length (Al-Masaeid et al.1995, Mawjoud and Sofia, 2008). Therefore, it is necessary to collect the details of tangent length of curve. Preceding tangent length (PTL) is the straight portion of road available before start of the curve. Preceding tangent length before speed observation point (PTLS) is also the straight portion of road available after the start of tangent and before speed observation point. In this study it is hypothesized that straight length before the speed observation point also may have the influence on tangent speed. The PTL was measured from the CAD drawing and PTLS was measured at the site.

3.4.4 Superelevation (e)

The transverse inclination of the roadway surface provided to counteract the effect of centrifugal force and reduce the tendency of the vehicle to overturn is known as superelevation. Superelevation is calculated by measuring the difference in levels on either side of the carriageway and the carriageway width. Superelevation provided at the start (e_S), middle (e_M), and end (e_E) of each selected curves are measured.

3.4.5 Sight Distance (SD)

The lack of proper sight distance accounts for about 8% - 10% of the accidents (Babkov, 1975). Short sight distances correspond with high accident frequency but also large sight distances might cause accidents (McCarthy, 2011).Therefore, sight distance is the important parameter to be considered in the design of a highway. Sight distance is the length of roadway ahead visible to the driver. It allows drivers to adjust vehicle controls in order to make safe movements and avoid possible obstructions. In this study, sight distance was determined by field observations as vegetations, sign boards, disabled vehicles or other obstacles which may interfere with available sight distance. Sight distance was measured, considering height of the driver's eyes, h_1=1.2 m and the height of the obstruction, h_2=0.15 m, as per IRC guidelines. A number of trials were performed to determine the available sight distance at the start (SD_{TS}) and middle of tangent point (SD_{TM}), and start (SD_S) and middle of the curve (SD_M).

3.4.6 Carriageway Width (W)

The width of carriageway at start (W_S), middle (W_M) and end (W_E) of each curve was measured using tape.

3.4.7 Shoulder Width(S)

The shoulder width is the clear unpaved surface available on either side of the roadway. The width of shoulder available at start (S_S), middle (S_M) and end (S_E) of each curve was measured on site using tape.

3.5 SPEED DATA

On the basis of the literature review (Chapter 2), it was decided to collect the spot speeds of passenger vehicles at different locations along a road. Registration number matching method was adopted so that the speed of the same vehicle is measured at different locations. It is, therefore, possible to monitor the effects of geometry on the speeds of individual vehicles, rather than trying to compare its effects on different populations of vehicles.

The operation speed on wet surface is found to be significantly slower than that on dry surface (Hong and Oguchi 2005). Therefore, the speeds were measured during daylight, off-peak periods, and under dry weather conditions. A total of 100-110 passenger cars were observed. The speeds were measured manually at all points during free-flow condition. Lamm et al. (1990) considered free-flow conditions to be isolated vehicles with a time gap of at least 6 seconds. Poe et al. (1996) assumed that the case of free-flow conditions would occur when the headways are equal to or greater than 5 seconds. In this study, to ensure that the measured speeds represented the free-flow speeds of isolated vehicles, a time gap between two successive vehicles greater than 25 sec was selected (Kanellaidis et al. 1990). To monitor the effect of curvature on speed, speed was collected at the tangent, start, middle, and end of the curve. At each point, a trap length of 10 m was considered and the time taken for each vehicle to traverse the trap was noted along with registration number. Observers were located at each point in such a way that their presence would not influence the speeds of passing vehicles. The details collected at four points on the horizontal curve are shown in Fig. 3.3 and the points are as follows:

- Point 1 (TM): on the tangent section and away from the approaching start point of the curve
- Point 2 (S): Start point of the curve
- Point 3(M): Middle of the curve
- Point 4 (E): End of the curve

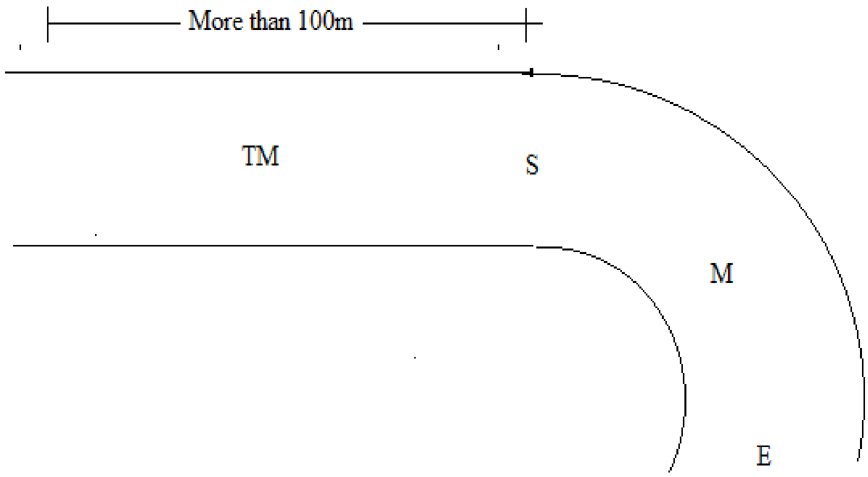


Fig. 3.3 Speed observation points on a typical Horizontal Curve

3.5.1 Operating Speed

Operating speed is defined as the speed selected by the highway users when not restricted by other users, i.e. free flow conditions, and is normally represented by the 85th percentile speed (V_{85}) (Fitzpatrick et al. 2000a). Operating speed is also defined as the speed that 85% of the drivers do not exceed (Gibreel et al.1999). The values of observed operating speeds (85th percentile speed) of horizontal curves at four different points, i.e. on midpoint of tangent section (V_{TM}), at starting point of curve (V_S), at middle of the curve (V_M), and the end point of the curve (V_E) were determined following the procedure given below and the values are tabulated in Appendix 1, Table A 1.1.

Time taken by each vehicle to traverse the trap and registration number were entered in a spreadsheet. Then spot speed of each vehicle was calculated. Any vehicle that had been tagged in the field for unusual behaviour was removed from the spreadsheet. A typical speed spreadsheet is given in Appendix 1, Table A1.2. Similarly, for each set of speed observations at four different points on each selected curve, a calculation spreadsheet was made use of. Then the observed speeds were evaluated for sufficiency of the sample size at 5 % significance level and allowing a permissible error in speed of ± 2 km/h in Eq.3.1. Table 3.4 presents the sample size sufficiency evaluation for H101.

Minimum number of vehicles' speed data to be collected i.e.

$$\text{Sample size} = \left[\frac{z_{a/2}\sigma}{E} \right]^2 \qquad \text{-- (3.1)}$$

Where,

a= 0.05 at 95% confidence level

$Z_{a/2} = Z_{0.05/2} = 1.96$

σ = Standard deviation of samples collected

E =Specified error

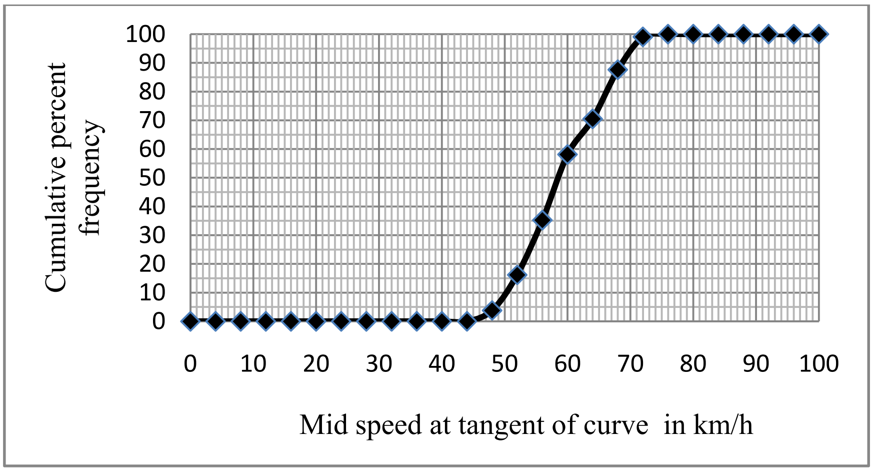
Table 3.4 Sample size sufficiency check

Point	Passenger cars	
	Collected	Required
Tangent	105	47
Start of curve		40
Mid of curve		40
End of curve		42

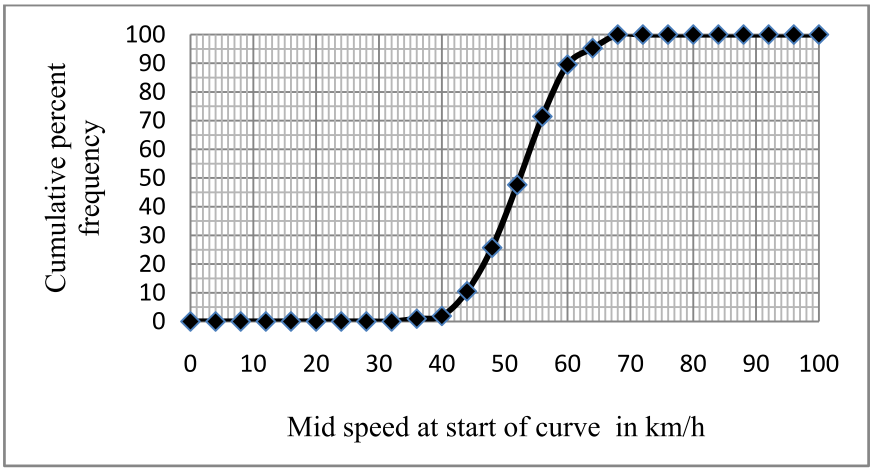
To obtain the operating speed, V_{85} cumulative percentage frequency distribution table was prepared, based on the observed speeds. The spot speed values were grouped using the class interval estimated as given in Eq 3.2.

$$\text{Class interval} = \frac{\text{Range of values}}{\left(1 + 3.22\log_{10}(\text{number of observations})\right)} \quad \text{----- (3.2)}$$

Class interval was found to be between 4 to10 km/h. The cumulative percent frequencies were estimated for each speed range. A sample of estimated cumulative percent frequencies are given in Appendix 1, Table A1.3. Cumulative percentage frequency plot as shown in Fig.3.4 was prepared. The speed value corresponding to 85th cumulative percent was found from this plot. Then the 85th percentile speeds, V_{85} was obtained from the cumulative percentage frequency graph. A typical cumulative percent frequency graph at different study points of horizontal curve are shown in Figs. 3.4(a) to (d).

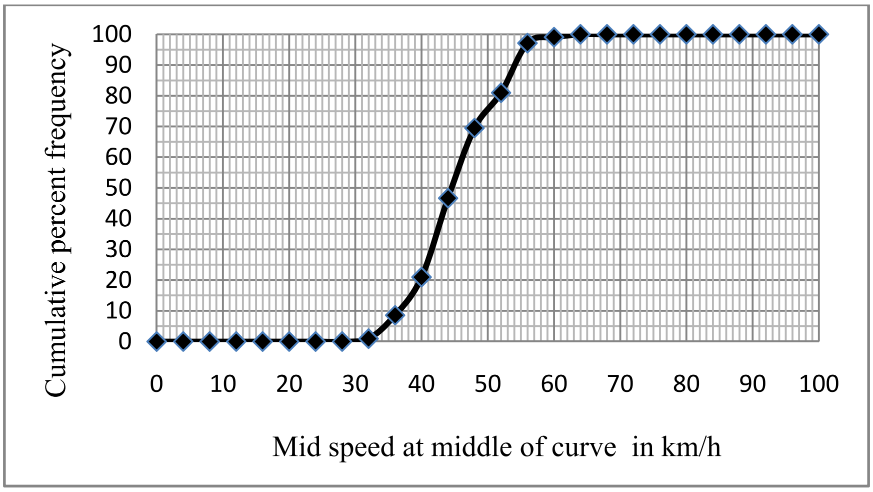


(a) Operating speed at tangent point of H101

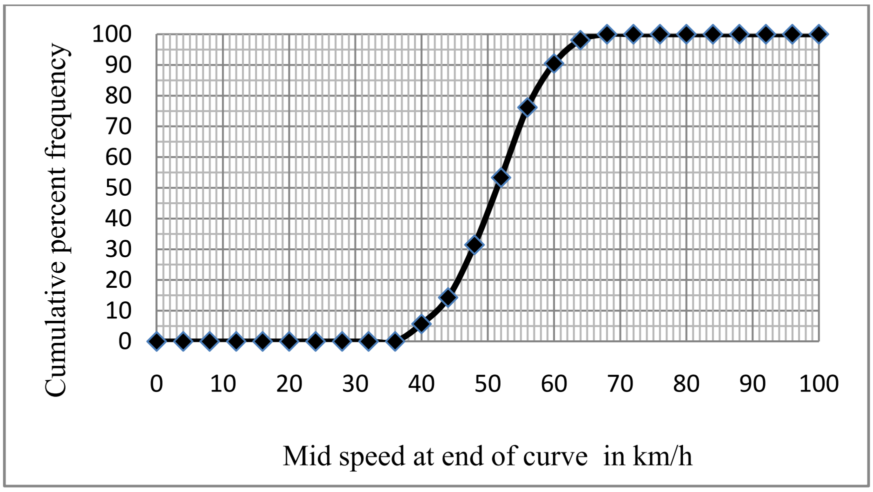


(b) Operating speed at start point of H101

Fig.3.4 Operating speed at study points



(c) Operating speed at middle point of H101



(d) Operating speed at end point of H101

Fig.3.4 Operating speed at study points

3.5.2 Trend of operating speed at study points

From Fig 3.5 it can be stated that the drivers tend to decrease their speed while approaching the midpoint of the curve, where the sight distance is supposed to be most limited and then increase as vehicle approaches the end of curve. A plausible explanation of this trend is that while moving from tangent section to the middle point of curve, the sight distance decreases and this sight distance restriction forces the drivers to reduce their speed. Also decrease of curvature makes steering of the vehicle difficult, which is another reason for decrease of speed. However, on approaching the end point of horizontal curve, the sight distance starts to increase and drivers tend to increase their speed.

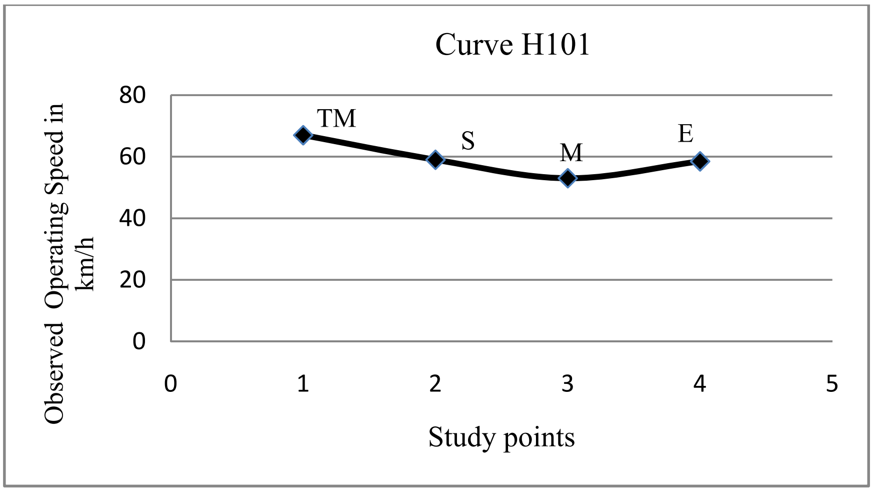


Fig.3.5 Trend of operating speed at study points

3.5.3 85th Percentile Speed Differential $(\Delta V)_{85}$

85th percentile speed differential is defined as the reduction in speed not exceeded by 85% of the drivers travelling under free flow conditions. This reflects the 85th percentile maximum speed reduction between two successive highway elements as experienced by the same vehicle or driver (Hirsh 1987, McFadden and Elefteriadou 2000, Misaghi and Hassan 2005). In this study, the values of observed speed differential between the study points were determined for each vehicle by matching speed with registration number of vehicles. For each set of speed observations at different points on each selected curve, a calculation spreadsheet was made use of. Then the observed speeds were evaluated for sufficiency of the sample size at 5 % significance level and allowing a permissible error in speed of ± 2 km/h using Eq 3.1. Table 3.5 presents the sample size sufficiency evaluation for horizontal curve 101 (H101).

Table 3.5 Sample size sufficiency check for speed differential

Speed differential between Point	Passenger cars	
	Collected	Required
Tangent and middle point of curve(TMM)	105	70

The spot reductions in speed values were grouped using the class interval. Class interval was calculated using Eq.2 and found to be 3 km/h in most of the cases. The cumulative percent frequencies were estimated. A sample of estimated cumulative percent frequencies are given in Appendix 1, Table A1.4. The speed differential value corresponding to 85th cumulative percent was found from cumulative percentage frequency plot. A typical cumulative percent frequency graph of speed differential between the tangent and middle point of curve (($\Delta V)_{85TMM}$) is as shown in Fig 3.6. The values of observed 85th speed differential of selected curves between tangent to middle $(\Delta V)_{85TMM}$ is tabulated in Appendix 1, Table A1.1.

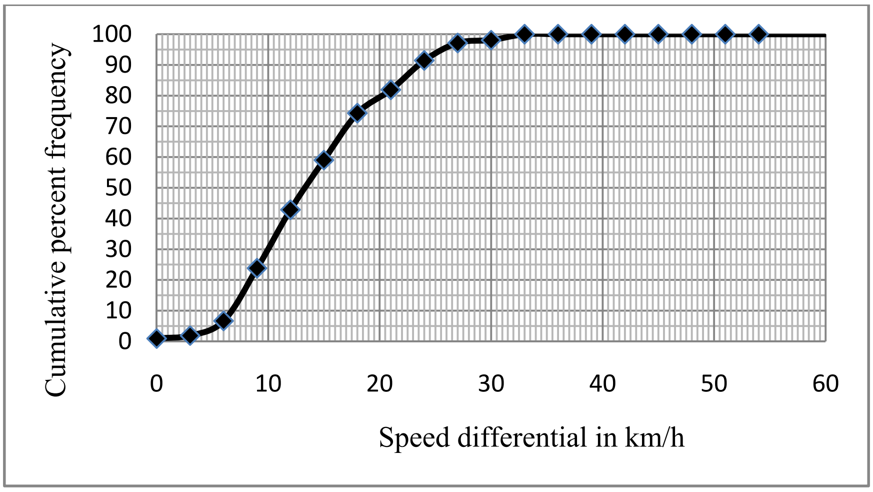


Fig. 3.6 85th Speed differential between tangent and midpoint of H101

3.6 ACCIDENT DATA

The only piece of information available for accident studies is the FIR (First Information Report) lodged in the police stations. It is difficult to have access to the accident particulars that are recorded in police diaries and files. With the prior permission of the concerned Superintendent of Police (S.P) and Sub Inspector of police (S.I), the details of accidents on the selected rural highways were collected from the 13 police stations. Accident details from 2005 to 2010 were extracted from FIRs filed under IPC NO.279, 337 and 338/304(A). The details such as property damage, simple injury, grievous and fatal injuries were noted down. Six years' accident data were collected year wise from each police station's records and then sorted out month wise, again from each month to particular day and listed. The types of vehicles involved in accidents recorded in FIRs were also noted down. The categories of vehicles include auto rickshaw, heavy vehicle, tempo, van, Jeep, car, two-wheelers etc. The types of injuries and the nature of severity of injuries that were inflicted from accidents i.e. simple, grievous, and fatal injuries were also noted. Number of pedestrians involved in accidents was also noted.

3.6.1 Identification of Accident Spots

Visit was undertaken along with a police official on the selected roads to identify accident spots. Help of at least two to three local persons was also obtained to identify the exact accident spots. Each accident was marked with colour coding to identify accidents of different severity levels on the map of the study area. Black spot diagram was drawn to obtain the number of accidents by type at each of the curves. Table A 1.1 in Appendix 1, gives the number of accidents by type at each of the curves.

The summary of data particulars collected of selected horizontal curves is tabulated in Table 3.6. The notations adopted are as follows:

H =Horizontal curves with tangent length more than 100 m with gradient -2% to +2%

e_S= Superelevation at the start of the curve in %

e_M= Superelevation at the middle of the curve in %

e_E=Superelevation at the end of the curve in %

SD_{TS}= Sight distance available at the start of tangent in m

SD_{TM}= Sight distance available at the middle of tangent in m

SD_S= Sight distance available at the start of curve in m

SD_M= Sight distance available at the middle of curve in m

W_S= Road width at start of the curve in m

W_M= Road width at middle of the curve in m

W_E= Road width at end of the curve in m

S_S= Shoulder width at start of the curve in m

S_M= Shoulder width at middle of the curve in m

S_E= Shoulder width at end of the curve in m

PTL = Preceding tangent length in m

PTLS= Preceding tangent length before speed observation point in m

$(\Delta V)_{85TMM}$ = Speed differential between the tangent and middle point of curve

Table 3.6 Summary of data collected

Number of horizontal curves collected: 178				
Observed values		Maximum	Minimum	Average
Radius of curve in m		635	29	175.2
Deflection angle in deg		98	8	36.0
Length of curve in m		263	35	96.9
PTL in m		423	100	164.4
PTLS in m		210	50	81.2
Superelevation in %	e_S	11.6	0	2.1
	e_M	13.3	0.14	4.5
	e_E	13.20	0.09	2.4
Sight distance in m	SD_{TS}	126.0	37.0	78.8
	SD_{TM}	120.0	32.0	66.7
	SD_S	126.0	19.0	43.4
	SD_M	83.0	18.0	48.9
Road width(m)	W_S	7.0	4.4	5.5
	W_M	7.1	4.7	5.6
	W_E	7.1	4.4	5.5
Shoulder width(m)	S_S	6.5	0.0	1.5
	S_M	7.7	0.0	1.6
	S_E	6.0	0.0	1.4
Operating speed in km/h	V_{TM}	73.0	47.0	62.9
	V_S	68.0	35.5	54.3
	V_M	60.0	30.0	46.8
	V_E	65.0	33.0	53.2
Speed differential in km/h	$(\Delta V)_{85TMM}$	35.0	11.0	22.0
Number of Accidents	Fatal	2	0	-
	Grievous Injury	5	0	-
	Simple Injury	3	0	-

3.7 EXPLORATION OF ACCIDENT DATA AT HORIZONTAL CURVES

Examination of accident data provided knowledge of accident variation and its distribution. The accident data collected for six years, from 2005 to 2010, was used for this purpose. During the pilot survey it was possible to identify locations of 222 accidents out of 290 reported accidents of last three years (2008-2010) i.e. nearly

76.5% of accident spots on the selected stretches. Out of that, 145 accidents occurred at horizontal curves with tangent length of more than 100 m. Table 3.7 shows the identified number of accidents on the selected stretches and horizontal curves.

Table 3.7 Identified accidents on the selected stretches and horizontal curves

Type of accidents	Fatal accidents	Grievous Injury	Simple Injury	Total number of accidents
Reported accidents	36	152	102	290
Identified accidents in the study stretch	30	133	59	222
Identified accidents in the horizontal curves with 100 m tangent	20	85	40	145

3.7.1 Analysis based on Yearly Variation of Accidents

Analysis of the identified accidents at the selected horizontal curves revealed that total accidents increased from 36 in 2008 to 64 in 2010, with an increase of 77.7% over three years as shown in Fig 3.7. This may be due to the increase in population and number of registered vehicles. The average annual increase of accidents is 33.6%. This accident rate is quite high and requires immediate attention.

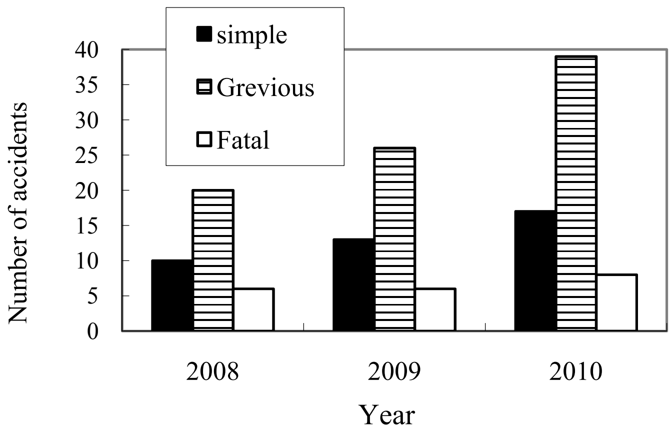


Fig.3.7 Distrubution of accidents based on type of injury

3.7.2 Analysis based on Month wise Variation of Accidents

The monthly variation of accidents at the horizontal curves during 2008-2010 is as shown in Fig.3.8.

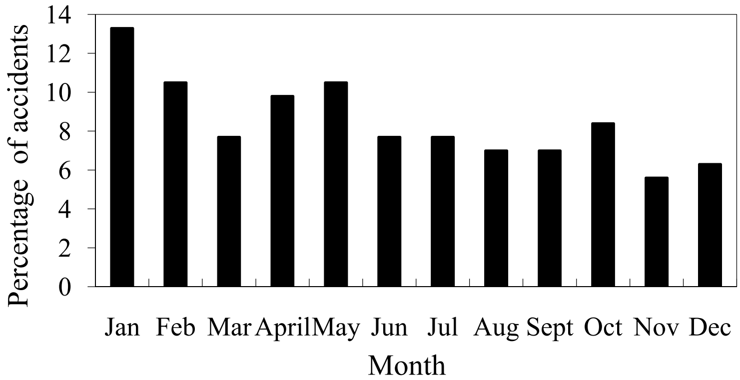


Fig.3.8 .Monthly variation in accidents

The results clearly show that the percentage of accidents is comparatively high in the early months of the year, followed by a decreasing trend up to March, and again showing an increasing trend up to the month of May, and then declining. Again, percentage of accidents shows an increasing trend in the month of October. An average of 13.2% of total accidents occurred in the month of January. This may be due to increased traffic on roads on account of travel by a large number of people to holy places like Shabarimala in the neighbouring state, and also to other places in the district to attend festivals in the month of January. As thousands of people from both Karnataka and outside visit tourist places and temples within the district during Dasara festival season, there is an alarming rise in accidents in October.

3.7.3 Analysis based on Time of Day Variation of Accidents

Hourly accidents at selected horizontal curves for the study are as shown in Fig.3.9. It reveals that nearly 35.1% of accidents occurred between 7am and 10 am and nearly 22.7% of accidents occurred between 5pm and 8 pm. The higher intensity of traffic flow during peak hours may be the reason.

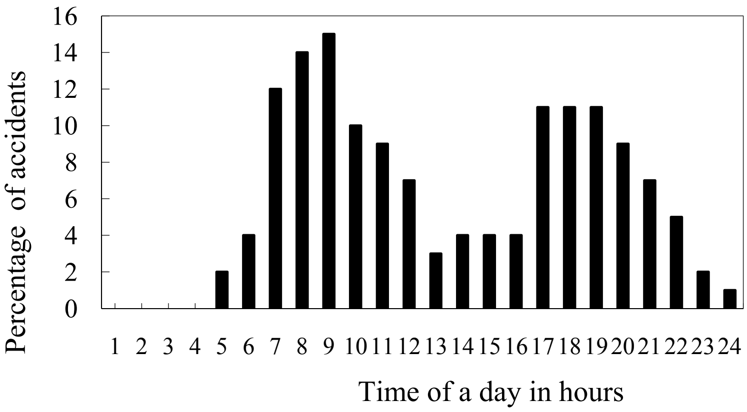


Fig. 3.9 Time of a day variation in accidents

3.7.4 Analysis based on Type of Vehicle Involved

Distribution of accidents by type of vehicle involved for the three years is as shown in Fig.3.10. During a period of three years, of the total number of vehicles involved in accidents, 29% were two-wheelers, 53% were four-wheelers, and 18% were heavy vehicles. Four-wheelers involved in accidents increased from 40 in 2008 to 60 in 2010, an increase of 60% in three years. This is again a disturbing trend and needs to be effectively tackled in the district.

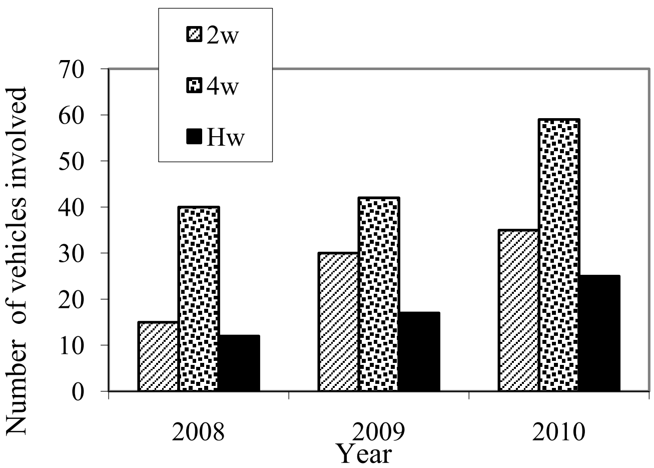


Fig.3.10. Number of vehicles involved in accidents

3.7.5 Analysis on Number of Accidents versus Radius

Fig.3.11 shows a plot of number of accidents and radius of curve. It can be observed that the number of accidents is more up to a radius of 400 m. The radius of less than

250 m is very dangerous because the number and severity of accidents are more. Also from this study, it was observed that 65.3% (145/222) of the total accidents on intermediate lane rural highways occurred on curved sections followed with straight tangent length of more than 100 m. Hence it can be concluded that curved sections and the corresponding transition sections represent the most critical locations while considering measures for improvement of highway safety.

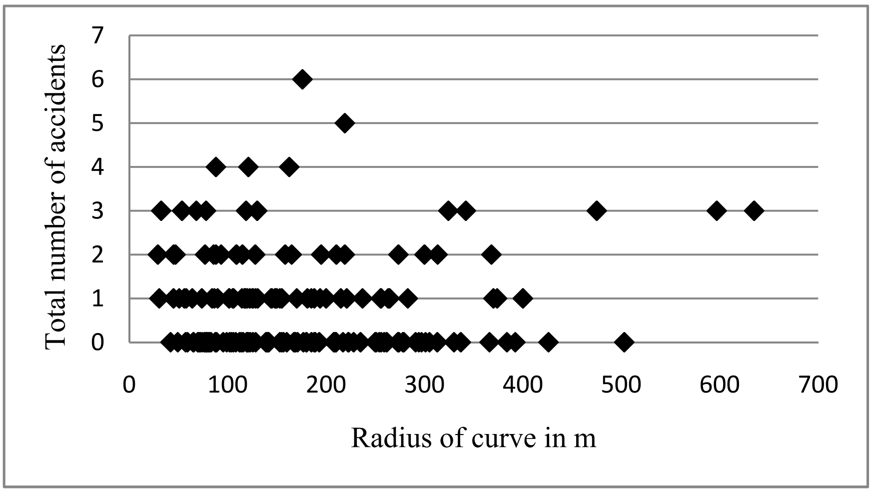


Fig.3.11 Relation between radius of curve and total number of accidents

3.8 DEVELOPMENT OF SPEED MODELS

As a large portion of collisions have been attributed to improper speed adaptation, operating speed can be a good indicator of the level of safety on the road segment (Nicholson 1998). One of the ways in which operating speeds are used in ensuring design consistency is through the use of operating speed models. In this study, both operating speed models and speed differential models are decided to estimate based on a function of the roadway geometry along the highway alignment. Thus, estimation of speed prediction models using the various components of the highway alignment is a key step in consistency evaluation. The main applications of speed models are:

- The operating speed/speed differential of a section can be estimated by knowing the geometric details of a particular location.
- During the design stage, estimated operating speed/speed differential can be used for evaluation of safety.

- Post construction, they can be used for rectification and improvement of geometrics.

3.8.1 Model Estimation

In this study, to predict operating speed and speed differential models, approximately two-thirds of data were used for estimation of model parameters and one-third of data were used for the validation of the models.

In order to identify the variables that can be as predictors and the nature of relationship, scatter plot and correlation analysis were carried out. Regression analysis was used to calibrate and determine if any relationship exists and fits the data well between the parameters in order to develop a model that will present the speed characteristics for the curves. The following conditions were adopted in the regression analysis to select the final models:

- The signs of the coefficients of the independent variables must have a logical explanation.
- The coefficient of determination (R^2) must be significant. The coefficient of determination is the percentage of variation of response variable explained by the regression. Higher the R^2 value, better the model is. The R^2 measures the proportionate reduction of total variation in the dependent variable associated with using a particular set of independent variables.
- The developed model should satisfy the overall significance of the regression (F-test) at the 0.95 confidence level.
- Each of the independent variables used in the model must have a regression coefficient that is significantly different from zero at the 0.95 confidence level. In this study, the variable having t-test value higher than two (20 degrees of freedom) was selected for model development.
- The developed model should have low value of Percentage Root Mean Squared Error (PRMSE).

3.8.2 Model Validation

Validation of selected models was carried out using the remaining one third of data. The following analysis was carried out for each of the speed predicted equation:

- The operating speed/85th speed differential value at each point was calculated using the respective model for that point. Scatter plots of predicted and observed values are superimposed with 45 degree line.
- The differences between the predicted and observed operating speeds/85th speed differential were calculated as:

  Error, e_i = (Observed value - Predicted value)^2
- The squared sum of errors (SSE) was calculated using

  SSE = $\sum e_i$
- The mean squared error (MSE) was calculated using

  MSE = SSE/n

  Where n = number of samples
- The root mean squared of errors (RMSE) was calculated using

  RMSE = $\sqrt{MSE}$
- The Percentage Root Mean Square Error (PRMSE) was calculated as

$$\text{PRMSE} = \frac{RMSE}{\overline{Y}} \times 100 \qquad \text{------- (3.3)}$$

Where ,

$\overline{Y}$ = Mean of the observed values

The model is said to be good when the PRMSE value is less.

3.8.3 Estimation of Operating Speed Model

The actual operating speed must correspond to the road design elements. Therefore, the details of geometric variables and observed speed on horizontal curves used for model development are tabulated in Appendix 1, Table A1.1. Out of 178 horizontal curves data, the data of 165 horizontal curves were used for the calibration and validation of operating speed models. Remaining 13 horizontal curves were found to be of dissimilar characteristics, hence not considered further. Initially, graphical analysis and correlation analysis were performed to determine the influences of geometric variables on speeds of passenger cars. Then stepwise linear regression was performed between the speed and the significant geometric variables at four study points, namely tangent section (V_{TM}) on starting point of the curve (V_S), middle of the curve (V_M), and end point of the curve (V_E), thereby identifying the variables which have high explanatory value.

Using scatter plots and correlation matrix between geometric variables and observed speeds at points of study, a number of independent variables were identified. Table 3.8 shows the best regression models developed using SPSS(Statistical Package for Social Sciences). It can be observed that radius is the main influencing variable and sight distance is another variable which affects the operating speed at study points.

Table 3.8 Models developed before classification of road

Location	Model	R2
Start point	VS= 48.543+1.848x 10-2 Rc +4.256x10-2 SDTM t=27.6 t=3.5 t =2.5	0.24
Middle point	VM=42.151+2.119x 10-2 Rc +1.499x10-2 SDTM t=27.0 t=4.2 t =2.3	0.21
End point	VE=45.783+1.622x 10-2 Rc +0.101 SDM t=27.9 t=3.2 t =2.5	0.26

Where ,

V_S , V_M and V_E are the operating speeds at start, mid and end of horizontal curves respectively, in km/h.

R_c = Radius of curve in m

SD_{TM} = Sight distance available at mid of tangent in m

SD_M = Sight distance available at mid of curve in m

The coefficient of determination (R^2) of model is very low and it varies between the range 0.21 - 0.26. It indicates that there is/are other variable/s which influences the operating speed. A critical observation of the scatter plots revealed that there are clusters of points with different trends. A number of variables were considered, one by one, to identify these clusters. Carriageway width at middle of curve was found to be responsible for these clusters.

Also, it was observed from literature that, the width of lane has been shown to be an important variable having relation with accidents (Fitzpatrick et al.2000a and Zegeer et al.1986). Results summarised by Gibreel et al. (1999) have consistently shown that both the risk of collision and collision rates are much higher on curved sections than on straight segments. In addition, drivers are found to adopt an operating speed based on their reading of the road features rather than the design speed or the posted speed (Fitzpatrick et al. 2003). Hence the curves were classified according to road width available at the middle of the horizontal curve.

Even though this study is on intermediate lane rural highways, in the field of the carriageway width at study points varies between 4.7 m and 7.1 m. There is no recommendation of extra widening as given by IRC for intermediate lane highways. IRC recommends extra widening to be provided at the middle of the curve of single lane with radius of 20-60 m is 0.6 m and nil for radius of curve more than 60 m. An empirical formula has been recommended by IRC for finding the total extra widening as:

$$We = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \qquad \text{-- (3.4)}$$

Where,

We = Total extra widening in m

n = Number of traffic lanes (taken as 1 for intermediate lane)

l = Length of wheel base of longest vehicle in m (taken as 6 m)

R_c = Radius of horizontal curve in m

V = Design speed (80 km/h for highways passing through rolling terrain)

At some curves, extra width required as per IRC Eq.3.4 is more than available in the field. Therefore, the classification of curves has been carried out for the data set, based on available extra width available in the field. Hence the horizontal curves having carriageway width at the centre of the curve 5.5-6.1m were separated from data set.

This process resulted in 155 horizontal curves out of which 2/3 (100curves) were used for model development and 1/3 (55 curves) were used for model validation. The correlation matrix was developed to identify tentative speed prediction variables. Scatter plots were also plotted to finalise the variables that were to be used in the model development. After examining these results, a simple linear regression analysis was performed to determine if one of the geometric variables was a significant predictor of operating speeds. The results of this analysis were further used in the multiple linear regression analysis.

3.8.3.1 At tangent point

From the scatter plot (Fig. 3.12 (a), (b)) and the correlation matrix (Table 3.9), it was observed that PTL and PTLS are the two variables having significant effect and similar correlation with operating speed at tangent point. PTL gives a platform for a driver to achieve the desired speed, hence compared to PTLS, PTL is a more relevant variable to predict the operating speed at tangent point.

The scatter plots show that V_{TM} increases with increase in preceding tangent length. The rate of increase in speed is more for an increase in tangent length up to 300 m and the rate diminishes beyond this. It is also clear from Fig.3.12 (e) to (h) and Table 3.9, that operating speed at tangent point has a nonlinear relation with preceding tangent length. The sight distances available at the beginning of the tangent (SD_{TS}) and at middle of the tangent (SD_{TM}) also have a significant linear relation with operating speed, V_{TM}.

Using these results, regression models were developed to finalise the predictor variables of operating speed at tangent point of curve on intermediate lane rural highways. Initially, linear regression was performed to select the variable to be considered for multi linear analysis. Several regression models were developed and some are listed in Table 3.10. While developing the models, it is assumed that the drivers perceive the complexity ahead on road at some distance, and also that the model 2 shows the higher influence on V_{TM}; therefore, the variable SD_{TS} was included in the analysis.

Even though all the variables in the model show 95% confidence level, some of the variables in the model fail to satisfy t-test condition; hence such models were rejected during the analysis. Among various models, Model 12 shows a higher correlation (coefficient of regression, R^2 = 0.59) to predict operating speed at tangent point of curve. Hence model 12 is selected for consistency evaluation at tangent point of intermediate lane rural highways.

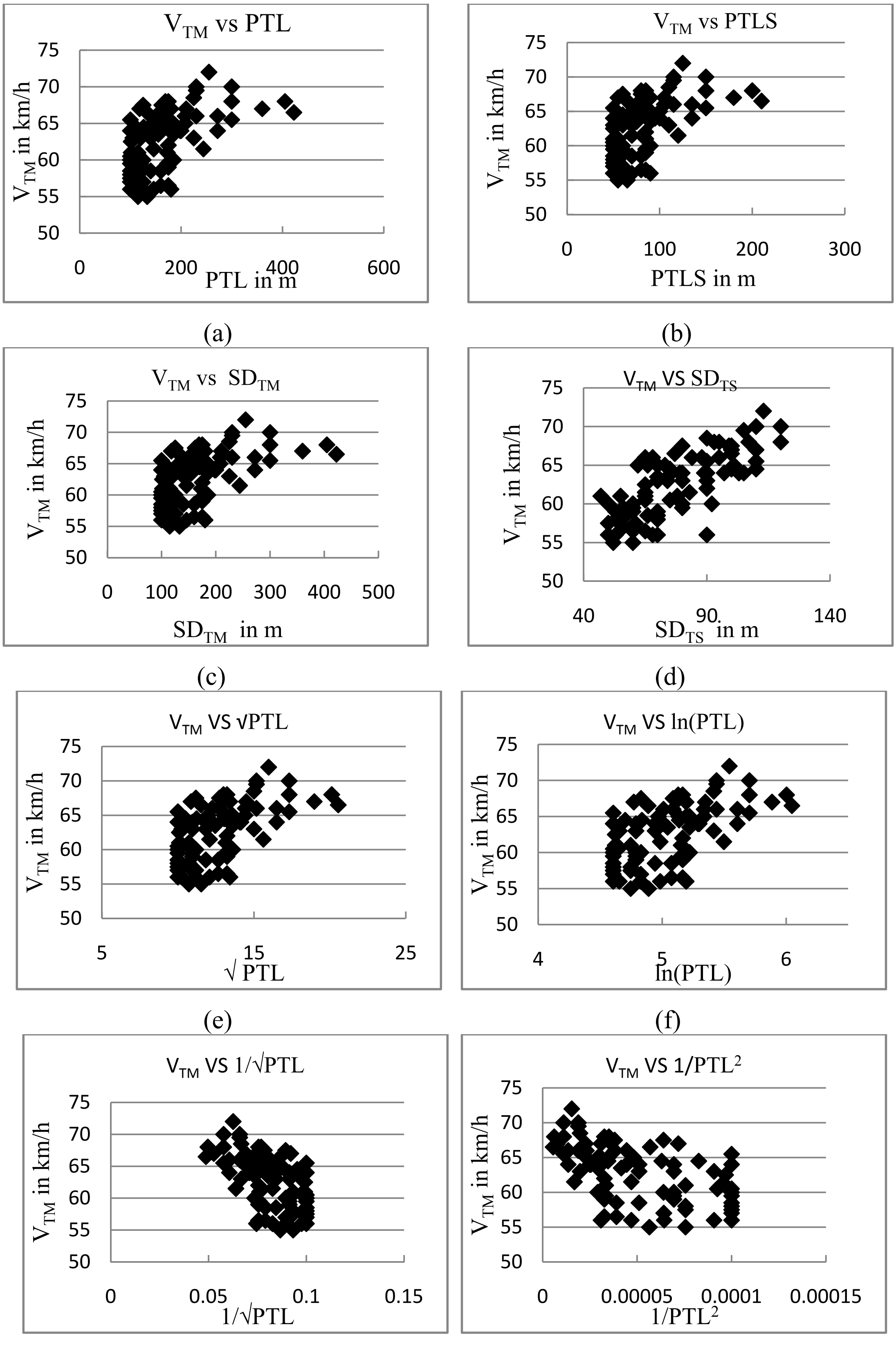


Fig.3.12. Relationship between operating speed at tangent point and geometric variables

Table 3.9 Correlation matrix developed at tangent point

t	PTL	PTLs	V_{TM}	SD_{TS}	SD_{TM}	√PTL	ln(PTL)	1/PTL^2	PTL^2	1/√PTL
PTL	1									
PTLs	1.00	1								
V_{TM}	0.55	0.55	1							
SD_{TS}	0.62	0.63	0.75	1						
SD_{TM}	0.60	0.60	0.69	0.80	1					
√PTL	-0.95	-0.94	-0.57	-0.61	-0.59	1				
ln(PTL)	0.97	0.97	0.57	0.62	0.60	-0.99	1			
1/PTL^2	-0.85	-0.84	-0.54	-0.54	-0.52	0.97	-0.94	1		
PTL^2	0.97	0.97	0.49	0.58	0.56	-0.85	0.90	-0.72	1	
1/√PTL	-0.95	-0.94	-0.57	-0.61	-0.59	1.00	-0.99	0.97	-0.85	1

The logic of model is that as the sight distance available at the beginning of the tangent (SD_{TS}) increases, the operating speed at tangent point (V_{TM}) also increases. Because of higher visibility with increase in preceding tangent length, drivers are able to view the oncoming feature in advance, and may accelerate their vehicles. The negative sign of inverse of √PTL indicates that as preceding tangent length increases the value of 1/√PTL decreases; therefore, the deduction value from constant reduces, which is reflected in higher operating speed with increase in PTL. Preceding tangent gives the plat form for the driver to accelerate the vehicles and to achieve the desired speed, hence having higher significance in predicting operating speed at tangent point. Therefore, it can be conclude that the speed on the tangent is found to be affected mainly by the tangent length and this is similar to the results of previous studies of Al-Masaeid et al. (1995).

3.8.3.2 Start point of curve

From both scatter plots (Figs. 3.13 (a) to (g)) and correlation matrix (Table 3.11), radius (R_c), inverse of the radius ($1/R_c$), length of curve (L_H) , sight distance available at mid of the tangent (SD_{TS}) and √R are found to be the most significant variables, but the relationship is not so good as expected.

Table 3.10 Regression models developed at tangent point of curve

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)							Coefficient of regression		F-value
		PTL	SD_{TS}	SD_{TM}	PLT^2	$1/PTL^2$	$1/\sqrt{PTL}$	ln(PTL)	R^2	Adjusted R^2	
1	57.117 (57.102)	0.33 (5.955)	-	-	-	-	-		0.299	0.291	35.45
2	49.775 (39.196)	-	0.163 (10.413)	-	-	-	-		0.566	0.561	108.43
3	52.132 (42.26)			0.159 (8.605)					0.471	0.465	74.04
4	29.107 (5.496)							6.646 (6.345)	0.327	0.319	40.26
5	76.747 (33.962)						-173.167 (-6.334)		0.326	0.318	40.11
6	66.509 (86.591)					-75609.9 (-5.819)			0.290	0.281	33.87
7	60.529 (106.3)				6.53×10^{-5} (5.117)				0.24	0.231	26.85
8	49.843 (39.464)	0.008 (1.425)	0.146 (7.335)						0.577	0.567	55.90
9	51.944 (41.902)	0.013 (2.157)		0.129 (5.734)					0.499	0.487	40.98
10	41.608 (9.16)		0.14 (7.12)					1.976 (1.87)	0.584	0.574	57.63
11	52.796 (28.316)		0.142 (7.74)			-25714 (-2.17)			0.587	0.58	57.00
12	56.108 (16.63)		0.139 (7.201)				-54.73 (2.02)		0.59	0.58	58.23

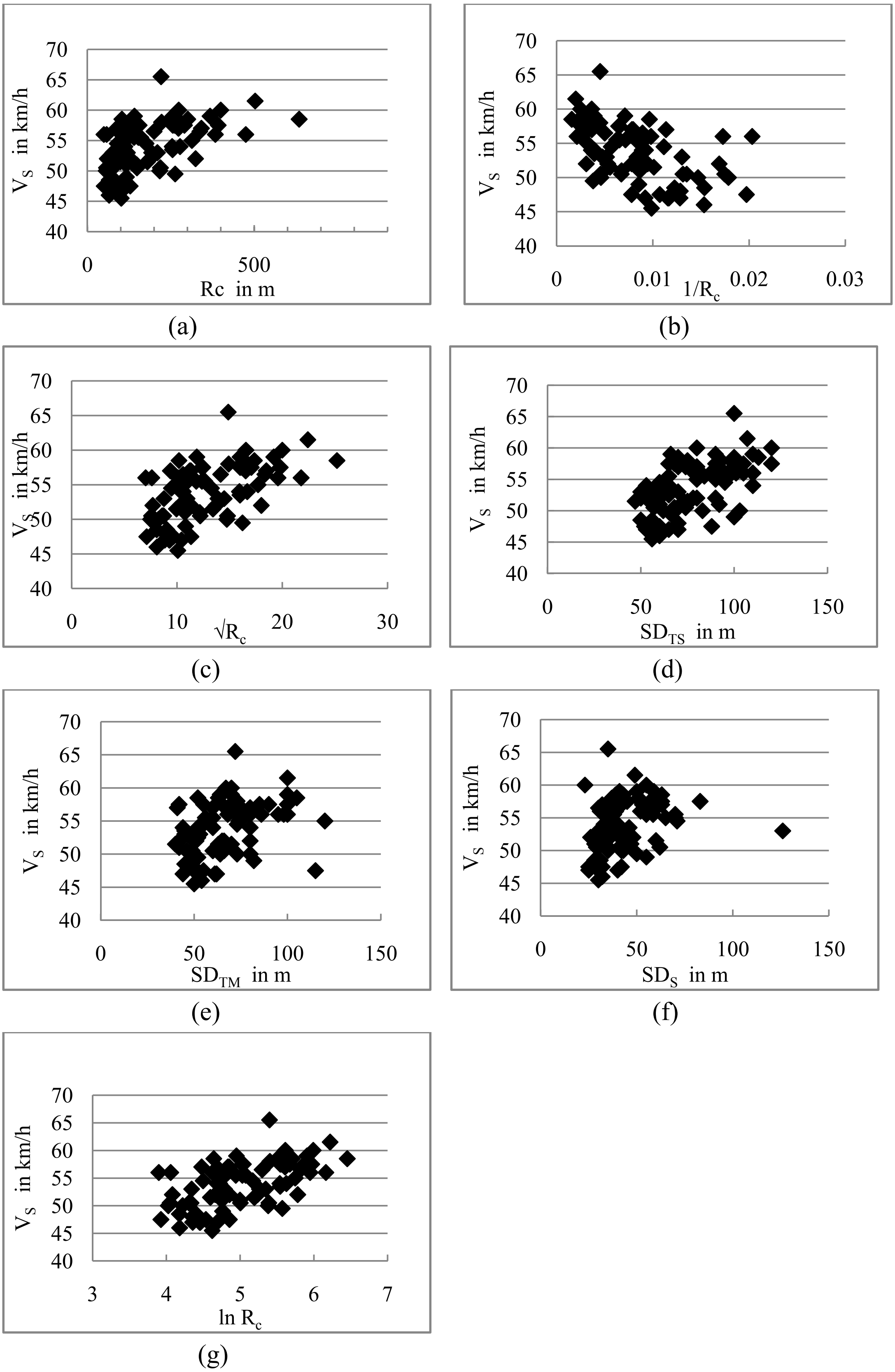


Fig.3.13 Relationship between operating speed at start point of curve and geometric variables.

Table 3.11 Correlation matrix developed for model development at curve

	R_c	$1/R_c$	Δ	L_H	PTL	V_S	V_M	V_E	e_S	e_M	e_E	SD_{TS}	SD_{TM}	SD_S	SD_M	S_S	S_M	S_E	$1/R_c^2$	$\sqrt{R_c}$
R_c	1																			
$1/R_c$	-0.81	1																		
Δ	-0.63	0.69	1																	
L_H	0.6	-0.59	0.02	1																
PTL	0.32	-0.16	0.02	0.41	1															
V_S	0.55	-0.54	-0.3	0.46	0.28	1														
V_M	0.58	-0.55	-0.34	0.49	0.2	0.79	1													
V_E	0.49	-0.55	-0.33	0.43	0.21	0.81	0.75	1												
e_S	0.07	-0.07	-0.14	-0	0.05	0.04	0.05	0.11	1											
e_M	-0.31	0.26	0.27	-0.2	-0.1	-0.1	-0.03	-0.06	0.11	1										
e_E	-0.22	0.11	0.09	-0	0.02	-0.23	-0.08	-0.04	0.06	0.33	1									
SD_{TS}	0.46	-0.34	0	0.62	0.62	0.56	0.38	0.44	0.04	-0.08	-0.12	1								
SD_{TM}	0.38	-0.21	0.04	0.48	0.6	0.38	0.2	0.24	0	-0.05	-0.15	0.8	1							
SD_S	0.14	-0.25	0.01	0.28	0.23	0.3	0.36	0.29	-0.1	0.3	0.01	0.4	0.35	1						
SD_M	0.25	-0.35	-0.14	0.34	0.35	0.39	0.35	0.44	-0.1	0.15	-0.01	0.53	0.44	0.64	1					
S_S	-0.04	-0.01	-0.05	0.01	0.2	0.05	-0.1	0.03	0.1	-0.09	-0.1	0.25	0.19	0.24	0.24	1				
S2	0.1	-0.11	-0.01	0.18	0.21	0.04	-0.11	0.04	0.15	-0.03	-0.13	0.37	0.29	0.23	0.25	0.79	1			
S3	0.12	-0.16	-0.07	0.18	0.2	0.04	-0.03	0.06	0.2	-0.02	-0.15	0.27	0.21	0.2	0.18	0.72	0.81	1		
$1/R_c^2$	-0.67	0.97	0.64	-0.5	-0.09	-0.45	-0.47	-0.5	-0.04	0.21	0.05	-0.23	-0.12	-0.27	-0.3	-0.03	-0.11	-0.17	1	
$\sqrt{R_c}$	-0.89	0.99	0.7	-0.6	-0.2	0.57	0.60	0.54	-0.08	0.29	0.15	-0.39	-0.26	-0.22	-0.3	0	-0.1	-0.15	0.92	1

For horizontal geometry on level and mild grades, the principal independent variable for operating speed at curve has generally been radius, R_c (Fitzpatrick et al. 2000a). In this study, it is also cleared that radius is one of the important variables influencing the operating speed at start point of curve. Figs. 3.13 (a) and (b) indicate that, as the radius of the curve increases, the $1/R_c$ value decreases, which reflects an increase of operating speed at start point of curve. Figs. 3.13 (d) to (f) shows that as sight distance available increases the operating speed at start of the curve also increases.

Drivers perceive the complexity ahead on road at some distance, before the speed observation point; hence, SD_{TS} and SD_{TM} are considered to predict operating speed at start of the curve. The scatter plots of other variables considered during the study were also plotted against operating speed at start of curve but fail to show the relationship to predict operating speed at start of the curve. After identifying the initial predictor variables from graphical analysis and correlation analysis, models are developed using regression analysis.

Table 3.12 presents the regression models developed at start point of intermediate lane rural highway curves. All the developed models show the logical explanation. It can be observed that all the variables satisfy the t-test, F-test conditions and show significance at 95% confidence level, but the coefficient of regression of models is less. Among various models developed, model 9 shows higher coefficients of regression and low PRMSE (5.6); hence, it is selected to predict the operating speed. $1/\sqrt{R_c}$ and SD_{TS} are the two variables in the model having significance to predict the speed at start point of curve.

The negative sign of $1/\sqrt{R_c}$ indicates that as radius of curve increases, the deduction value of the variable $1/\sqrt{R_c}$ decreases resulting in increase of operating speed at start point of curve. Also, as the sight distance available at the tangent point (SD_{TS}) increases, the complexity ahead due to introduction of curve decreases, hence the operating speed at start point of curve increases.

Table 3.12 Operating speed models at start point of curve of intermediate lane rural highways

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)						Coefficient of regression		F-value
		R_c	$1/R_c$	SD_{TS}	SD_{TM}	$1/\sqrt{R_c}$	$1/R_c^2$	R^2	Adjusted R^2	
1	50.282 (72.401)	0.019 (5.987)						0.302	0.293	35.84
2	57.606 (75.95)		-486.210 (-5.984)					0.287	0.279	33.46
3	55.453 (103.83)						-20372 (-4.630)	0.205	0.196	21.46
4	61.656 (46.892)					-92.461 (-6.227)		0.318	0.31	38.71
5	44.371 (28.31)			0.120 (6.18)				0.316	0.307	38.26
6	48.034 (30.47)				0.087 (3.784)			0.147	0.137	14.32
7	49.369 (27.458)		-354.374 (-4.491)	0.091 (4.939)				0.451	0.437	33.64
8	52.940 (31.97)		-431.714 (-5.28)		0.064 (3.13)			0.363	0.348	23.40
9	52.704 (23.051)			0.086 (4.572)		-66.824 (-4.619)		0.457	0.441	34.48
10	56.929 (27.045)				0.057 (2.808)	-81.603 (-5.52)		0.378	0.363	24.94

3.8.3.3 Midpoint of curve

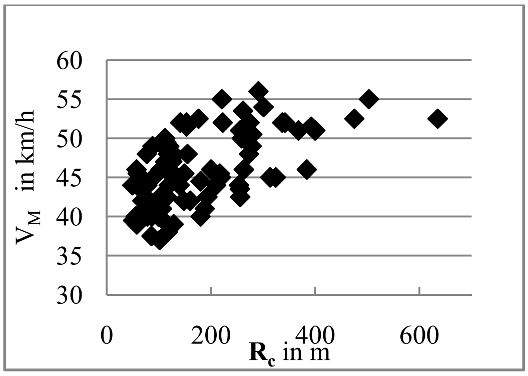
From the scatter plots (Figs. 3.14 (a) to (e)) and correlation matrix (Table 3.11) developed, it is observed that operating speed at midpoint of the curve (V_M) is more dependent on $\sqrt{R_c}$ than other variables considered. It can be observed from the Fig. 3.14 (a), that the operating speeds on horizontal curves are very similar to speeds on long tangents when the radius is approximately 400 m or more. The operating speed on horizontal curves drops sharply when the radius is less than 250 m.

Other variables such as R_c, $1/R_c$, Δ, $L_{H,}$ $SD_{s,}$ and R_c^2 also show a significant relationship with V_M. Using the results of scatter plots and correlation matrix developed, several models are developed and in which satisfy the t-test, F-test conditions and are significant at 95% confidence level, the logical explanations are listed in Table 3.13. The positive sign of R_c (model 1) indicate that as radius increases, the complexity during the driving decreases, and hence the speed increases. The positive sign of the SD_S (model 3) indicates that due to higher visibility drivers may accelerate their vehicles. The negative sign of the coefficients of variables $1/\sqrt{R_c}$ and $1/R_c^2$ indicates that as the variable increases, the operating speed at midpoint of curve decreases.

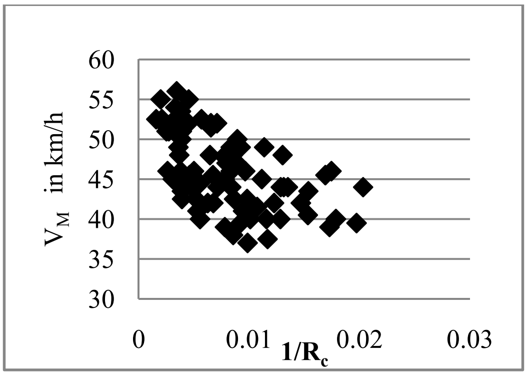
Among various models presented in Table 3.13, the model 12 shows higher coefficient of regression (R^2=0.43) and PRMSE of 8.4. Even though the model 11 shows coefficient of regression (R^2=0.39) and has a PRMSE of 7.8, hence, it is considered to predict operating speed at midpoint of horizontal curve. In the model 11, it is observed that the variable $1/\sqrt{R_c}$ does have more influence on operating speeds than the variable $1/R_c$. The negative sign of $1/\sqrt{R_c}$ in the model indicates that as the radius increases, the value of $1/\sqrt{R_c}$ reduces, thereby showing an increase in operating speed with increase of R_c.

3.8.3.4 End point of curve

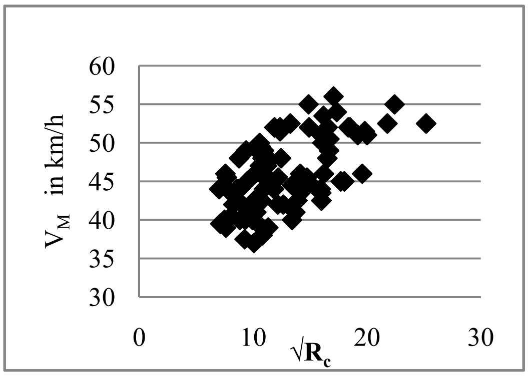
From scatter plots (Figs.3.15 (a) to (e)) and correlation matrix (Table 3.11) it can be found that $\sqrt{R_c}$ and sight distance (SD_M) are the most significant variables to predict operating speed at end point of horizontal curve (V_E).



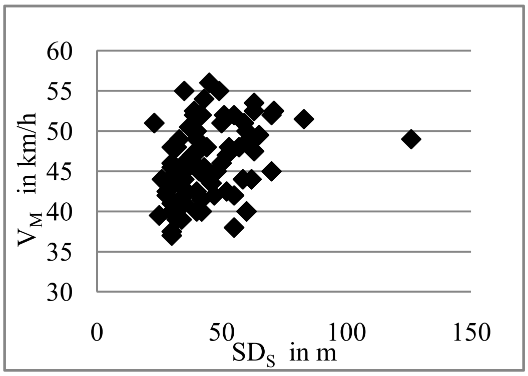
(a)



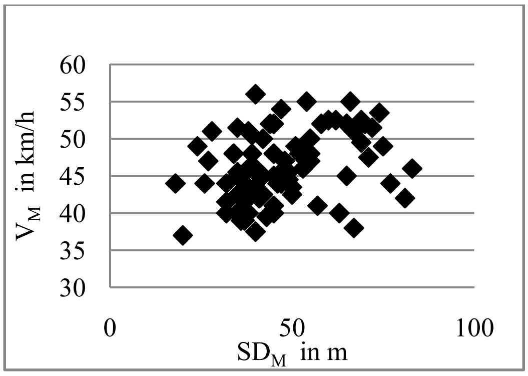
(b)



(c)



(d)



(e)

Fig.3.14 Relationship between operating speed at midpoint of curve and geometric variables

Drivers perceive the complexity ahead on road at some distance before the speed observation point, hence SD_M is considered to predict operating speed at end point of the curve. Among various models developed, Table 3.14 presents the models that satisfy the t-test and F-test and are significant at 95% confidence level at end point of curve.

Table 3.13 Operating speed models at midpoint of curve of intermediate lane rural highways

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)							Coefficient of regression		F-value
		R_c	$1/R_c$	Δ	L_H	SD_s	$1/\sqrt{R_c}$	$1/R_c^{\ 2}$	R^2	Ra^2	
1	41.801 (54.003)	0.024 (6.53)							0.339	0.331	42.64
2	50.525 (58.596)		-568.077 (-5.945)						0.299	0.290	35.34
3	41.365 (29.28)					0.108 (3.54)			0.131	0.12	12.51
4	49.961 (38.769)			-0.115 (-3.253)					0.113	0.102	10.58
5	40.865 (37.057)				0.056 (5.156)				0.243	0.233	26.58
6	48.041 (79.26)							24205.8 (-4.85)	0.221	0.211	23.52
7	55.277 (37.069)						-108.273 (-6.429)		0.332	0.324	41.34
8	53.736 (35.321)		-144.119 (-4.023)					44928.03 (2.526)	0.345	0.333	22.00
9	50.077 (26.87)		-1498.01 (-4.38)			0.082 (3.115)		51276.13 (3.01)	0.419	0.397	19.46
10	46.887 (29.19)		-506.263 (-5.319)			0.072 (2.64)			0.354	0.338	22.46
11	51.208 (24.977)					0.073 (2.77)	-98.002 (-5.898)		0.390	0.375	26.196
12	60.856 (13.465)		1394.29 (2.37)			0.084 (3.22)	-345.46 (-3.28)		0.43	0.409	20.34

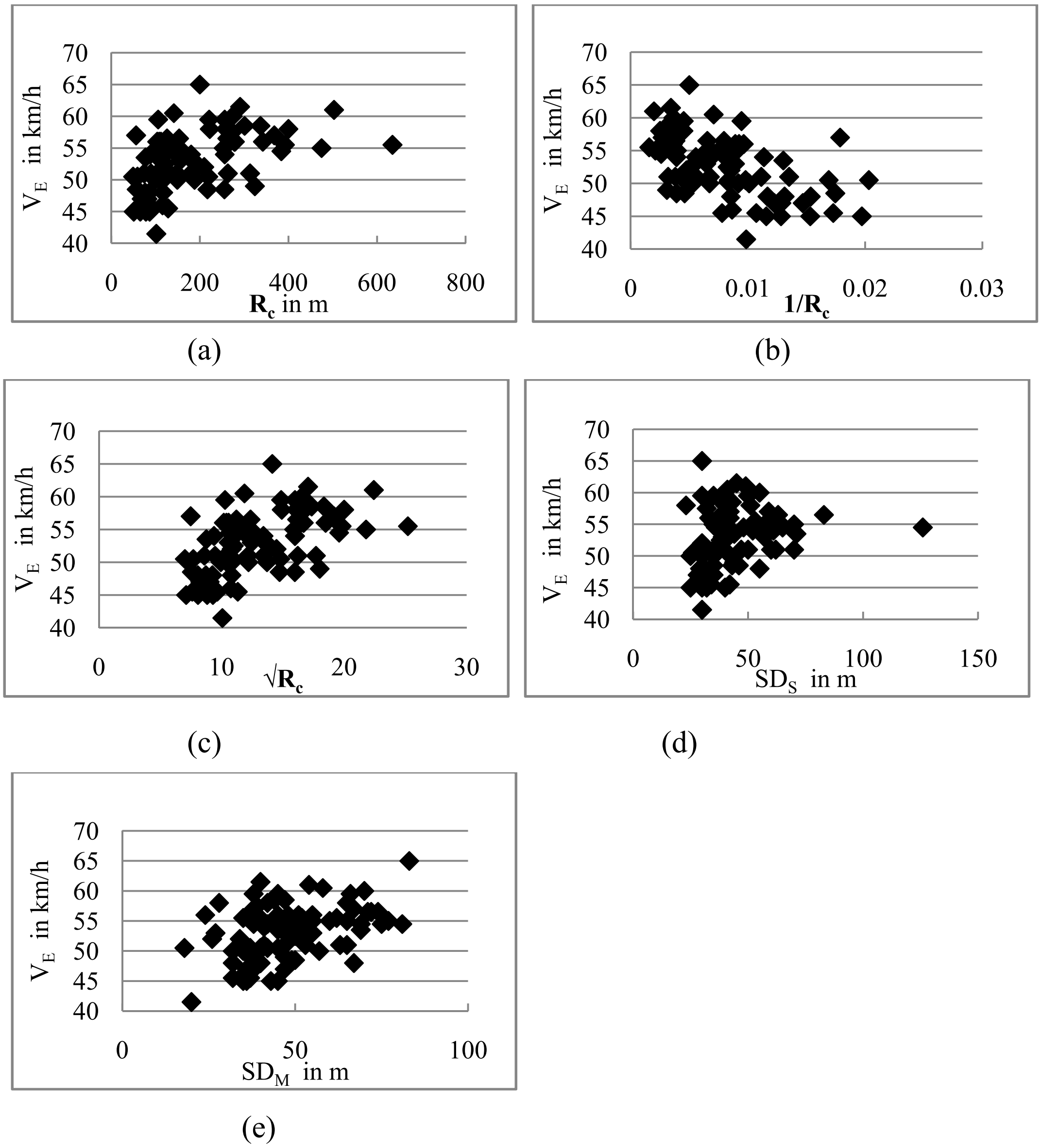


Fig.3.15 Relationship between operating speed at end point of curve and geometric variables

In the model 6, have the coefficient of regression (R^2) is 0.319 with PRMSE of 7.3 and in the model 8, inclusion of sight distance SD_M increases the coefficient of regression (R^2) value to 0.388 and changes PRMSE to 6.9. The model 8 shows higher coefficient of regression (R^2) among the various models developed, and hence is listed to finalise the model to predict the operating speed at end point of curve. The models logic is that as the radius of curve and sight distance available at end point of curve increase, the operating speed at end point of curve also increases.

Table 3.14 Operating speed models at end point of curve of intermediate lane rural highways

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)							Coefficient of regression		F-value
		R_c	$1/R_c$	Δ	L_H	SD_M	$1/\sqrt{R_c}$	$1/R_c^2$	R^2	Adjusted R^2	
1	49.203 (59.53)	0.20 (5.22)							0.241	0.238	27.27
2	57.363 (66.879)		-575.722 (-6.056)						0.306	0.298	36.68
3	56.692 (43.904)			-0.114 (-3.206)					0.110	0.099	10.28
4	48.327 (42.179)				0.048 (4.311)				0.183	0.173	18.58
5	46.090 (28.984)					0.142 (9.439)			0.192	0.182	19.70
6	61.877 (41.082)						-106.171 (-6.24)		0.319	0.311	38.96
7	54.932 (92.127)							-25583.1 (-5.211)	0.241	0.251	27.15
8	56.080 (23.394)					0.09 (3.022)	-88.403 (-5.12)		0.388	0.373	25.93
9	49.777 (29.09)					0.099 (3.193)		-20308.8 (-4.12)	0.33	0.313	20.17
10	52.274 (27.6)		474.406 (-4.893)			0.09 (2.985)			0.374	0.359	24.54

Table 3.15 lists the best models obtained from the regression analysis. The low PRMSE value (below 10) of the models indicates that the operating speed models are good enough in predicting the operating speeds at all study points. But coefficients of regression of all the models developed at start, mid and end point of curve are less; hence the application of these models during the design consistency evaluation is questionable.

Table 3.15 Operating speed models selected for intermediate lane rural highways

Class	Point at curve	Model	R^2	PRMSE	
				Calibration	Validation
5.5-6.1m road width	Tangent	$V_{TM} = 56.108 + 0.139\,SD_{TS} - \frac{54.731}{\sqrt{PTL}}$	0.59	4.2	6.4
	Start	$V_S = 52.704 + 0.086\,SD_{TS} - \frac{66.824}{\sqrt{R_c}}$	0.457	5.6	8.6
	Middle	$V_M = 51.208 + 0.084\,SDs - \frac{98.002}{\sqrt{R_c}}$	0.390	7.8	9.7
	End	$V_E = 56.080 + 0.09\,SD_M - \frac{88.4031}{\sqrt{R_c}}$	0.388	6.9	8.2

3.9 MODELS DEVELOPED BASED ON CLASSIFICATION

Models developed during earlier case are based on the assumption that operating speed on horizontal curves of intermediate lane rural highways is affected by carriageway width available at the middle of the curve. Zegeer et al. (1986) identified the influence of shoulder width on crash prediction and Lamm et al. (1988) recognized shoulder width as another geometric variable that influences the operating speed of the vehicle on horizontal curve. Hence the curves are again classified based on shoulder width available at the midpoint of horizontal curves. The main aim of this classification is to predict better significant operating models at selected observation points. Table 3.16 shows the number of curves in each class based on shoulder width available at the middle of horizontal curve.

Table 3.16 Classification of curves based on shoulder width

Class	Classification	No. of Horizontal curves
	Shoulder width	
A	0-1m	65
B	1-2m	45
C	2-3m	35

The number of curves with >3 m shoulder width are less and hence not considered for analysis.

3.9.1 Development of Models for Class A Curves

Based on the results of scatter plots and correlation matrix, the variables which show a good relationship with operating speed at study points were used to develop regression models. In this case, out of 65 curves, the data of 44 curves were used for calibration and data of 21 curves were used for validation.

3.9.1.1 Start point of curve

From scatter plots, shown in Figs.3.25 (a) to (f), it is observed that operating speed at start point of curve shows a good relationship with radius, deflection angle, inverse of radius, length of curve, nonlinear forms of radius considered and sight distance available at start of tangent. Correlation matrix developed for Class A curves is given in Appendix 2, Table A2.1. It is clear from the Table A2.1 that compared to linear form of radius of curve, non linear form of radius shows a better correlation with operating speed at start point of curve. Linear analysis was performed by taking each significant variable at a time and using these results, many trials were performed to develop multiple regression models by combining various variables. As numbers of variables used in the model increased, the R^2 value also increased, but many failed due to satisfy the criteria of model development.

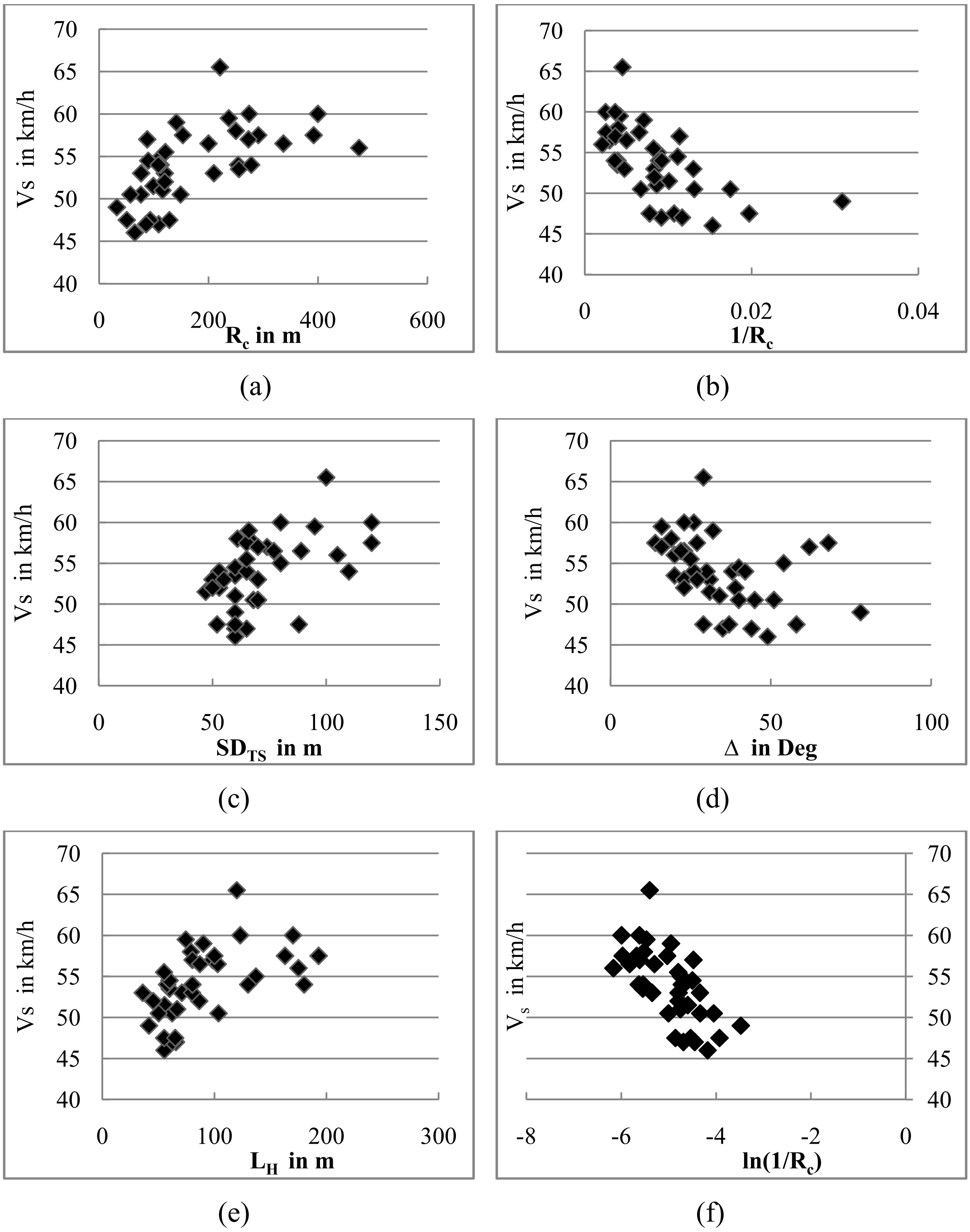


Fig.3.16 Relationship between operating speed at start point of Class A curves and geometric variables

Table 3.17 lists the models that satisfy all the conditions of model development and have logical explanation. The model 5 is found to be better and has more logical explanation than other models developed with higher coefficient of regression (R^2=

0.505) and low PRMSE (5.7); hence considered for consistency evaluation of start point of Class A curves.

It is clear from the model 5 that sight distance available at tangent point along with the radius affects the speed of passenger vehicles at start of the horizontal curve. V_S is related to logarithmic scale of inverse of radius, i.e. as radius increases, the logarithmic value of inverse of radius ($\ln(1/R_c)$) increases, resulting in higher speed at start point with increases of radius. However, vehicles approaching the start point of curve reduce their speed, because of reduced curvature. The model's logic is that increase of sight distance available reduces the workload of driver and creates the condition to accelerate the vehicle, but decrease of radius forces them to reduce the speed.

Table 3.17 Operating speed models at start point of Class A curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_S = 45.382 + 0.019R_c + 0.071SD_{TS}$ (t=21.42) (t=3.48) (t=2.26)	0.445	0.415	14.46
2	$V_S = 51.423 - \frac{380.316}{R_c} + 0.079SD_{TS}$ (t=19.75) (t=3.78) (t=2.63)	0.469	0.439	15.88
3	$V_S = 41.74 + 0.0586\sqrt{R_c} + 0.006SD_{TS}$ (t=18.52) (t=3.95) (t=2.17)	0.482	0.454	16.77
4	$V_S = 46.476 + 3.16x10^{-5}R_c^{\ 2} + 0.085SD_{TS}$ (t=20.28) (t=2.53) (t=2.56)	0.370	0.335	10.58
5	$V_S = 30.117 - 3.841n\left(\frac{1}{R_c}\right) + 0.065SD_{TS}$ (t=7.28) (t= -4.23) (t=2.17)	0.505	0.478	14.38

3.9.1.2 Midpoint of curve

From the Figs.3.17 (a) to (f) and Table A2.1, it is also clear that operating speed at midpoint of Class A curves shows a good relationship with geometric variables such as radius, inverse of radius ($1/R_c$) , deflection angle, length of curve, sight distance available at start of curve, $\sqrt{R_c}$, $R_c^{\ 2}$ and $\ln(1/R_c)$.

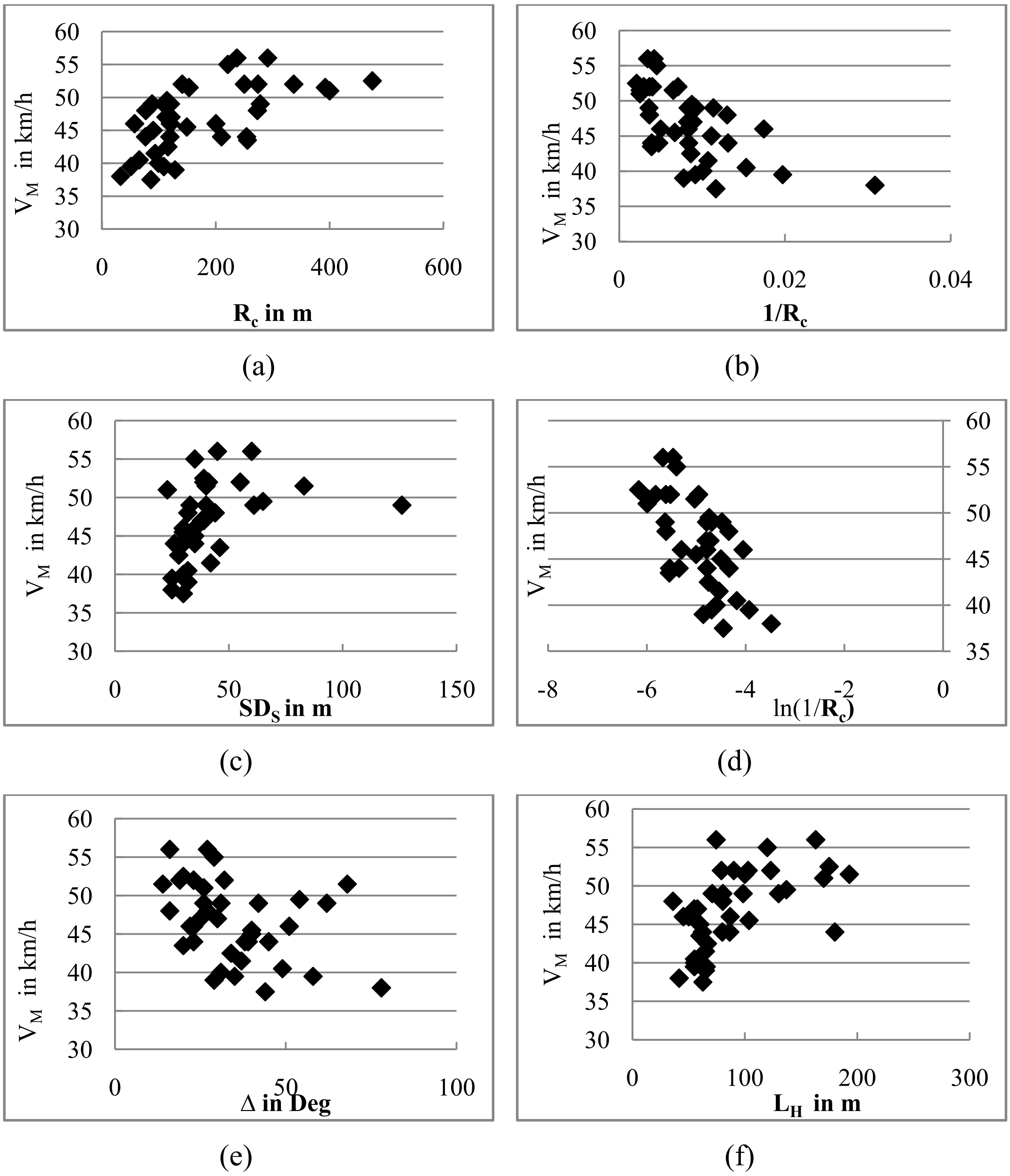


Fig.3.17 Relationship between operating speed at midpoint of Class A curves and geometric variables

Even though inclusion of sight distance available at start and mid of tangent increases the regression coefficient, sight distance available before the study point (SD_S) has more logical explanation than others, and hence considered for the analysis. Table 3.18 lists the multi linear regression models developed using the results of linear regression. All the explanatory variables are significant at 95% confidence level and are different from zero. Among various models developed, model 4 shows higher coefficient of regression and low PRMSE (7.1), and hence is considered for consistency evaluation of midpoint of Class A curves of intermediate lane rural highways.

Model 5 in Table 3.17 and Model 4 in Table 3.18 have similar variables; hence Model 4 in Table 3.18 has a similar logical explanation. Comparing these two models, it is clear that the constant value in the model decreases at midpoint of curve, which explains the deceleration of vehicle while approaching the midpoint of the curve. The logic of model 4 is that as radius of curve decreases, it makes the steering of vehicle at midpoint of curve is difficult; hence the speed at mid point reduces to a minimum value.

Table 3.18 Operating speed models at midpoint of Class A curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_M = 47.463 - \frac{505.834}{R_c} + 0.087SD_S$ (t=25.71) (t= -4.56) (t=2.61)	0.471	0.441	16.00
2	$V_M = 32.222 + 0.0816\sqrt{R_c} + 0.105SD_S$ (t=14.46) (t=5.64) (t=3.49)	0.555	0.530	22.42
3	$V_M = 39.677 + 5.63x10^{-5} R_c^{2} + 0.117SD_S$ (t=25.43) (t=4.6) (t=3.6)	0.475	0.445	16.26
4	$V_M = 17.383 - 5.109 \ln\left(\frac{1}{R_c}\right) + 0.098SD_S$ (t=3.75) (t= -5.61) (t=3.24)	0.557	0.533	22.65

3.9.1.3 End point of curve

From the results of scatter plots (Fig.3.18) and correlation matrix (Table A2.1), several linear and multi linear regression models are developed and, among them, most significant multi linear regression models are presented in Table 3.19.

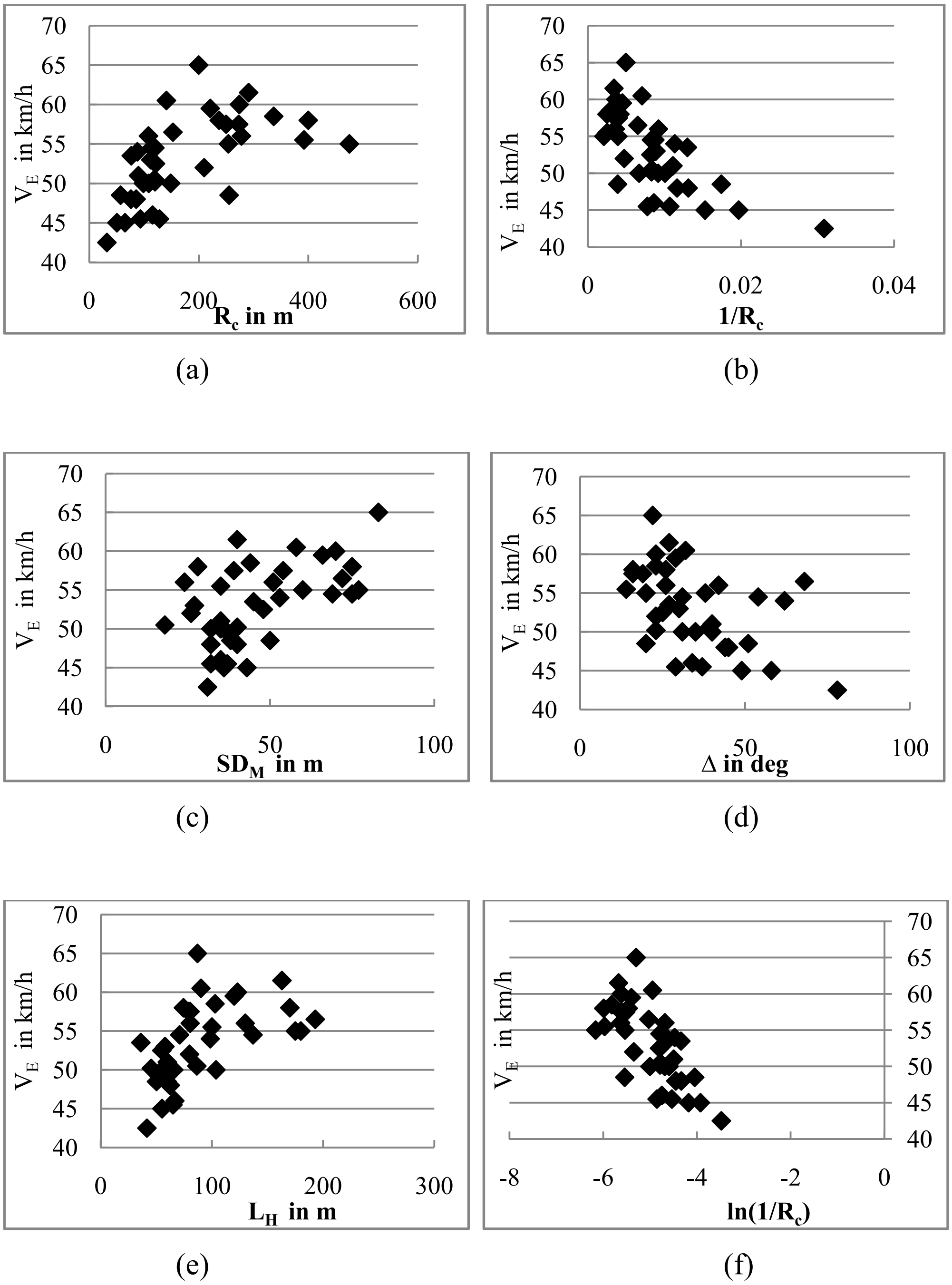


Fig.3.18 Relationship between operating speed at end point of Class A curves and geometric variables

Comparing the Model 4 in Table 3.18 with Model 4 in Table 3.19, it is clear that vehicles accelerate from the middle of curve to towards end of curve. When the vehicles approach the end point of curve, the complexity ahead to the driver decreases because of increased visibility. Therefore, drivers choose their safe speed after negotiating the midpoint of curve, which results in acceleration of vehicles after the midpoint of curve.

Table 3.19 Operating speed models at end point of Class A curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_E = 52.267 - \frac{540.534}{R_c} + 0.115SD_M$ (t=24.35) (t= -5.11) (t=3.27)	0.596	0.574	26.59
2	$V_E = 38.032 + 0.738\sqrt{R_c} + 0.124SD_M$ (t=17.03) (t= 4.76) (t=3.47)	0.573	0.549	24.17
3	$V_E = 44.263 + 4.22x10^{-5} R_c^{\ 2} + 0.153SD_M$ (t=23.32) (t= 3.21) (t=3.93)	0.459	0.429	15.26
4	$V_E = 23.146 - 4.948 \ln\left(\frac{1}{R_c}\right) + 0.115SD_M$ (t=5.16) (t= -5.23) (t=3.31)	0.605	0.583	27.602

It can be observed that the operating speed at the end of curve is affected by both radius and sight distance available at midpoint of curve. All the models show significance at 95% confidence level and satisfy the model development criteria. Among these models, model 4 show higher coefficient of regression (R^2 = 0.605) and low PRMSE (6.2)

3.9.2 Models Developed For Class B curves

Class B refers to the horizontal curve with 5.5 - 6.1 m carriageway width and 1-2 m shoulder width at the middle of curve. Out of 45 horizontal curves of Class B, the geometric and speed data of 32 curves were used for model development and the remaining are used for validation. Based on the results of graphical and correlation analysis, the variables to be considered in regression analysis were identified for all study points of curve.

3.9.2.1 At start point of curve

From scatter plots, given in Fig.3.19 and correlation matrix presented in Table A2.2, it is found that the operating speed at the start point of Class B curves shows good relationship with variables R_c, $1/R_c$, Δ, SD_{TS}, SD_{TM}, $\sqrt{R_c}$ and R_c^2. Sight distance available at start of the tangent (SD_{TS}) shows a higher influence to predict operating speed at start point of curve than the other variables considered. Among various regression models developed, the models having a higher coefficient of regression are presented in Table 3.20.

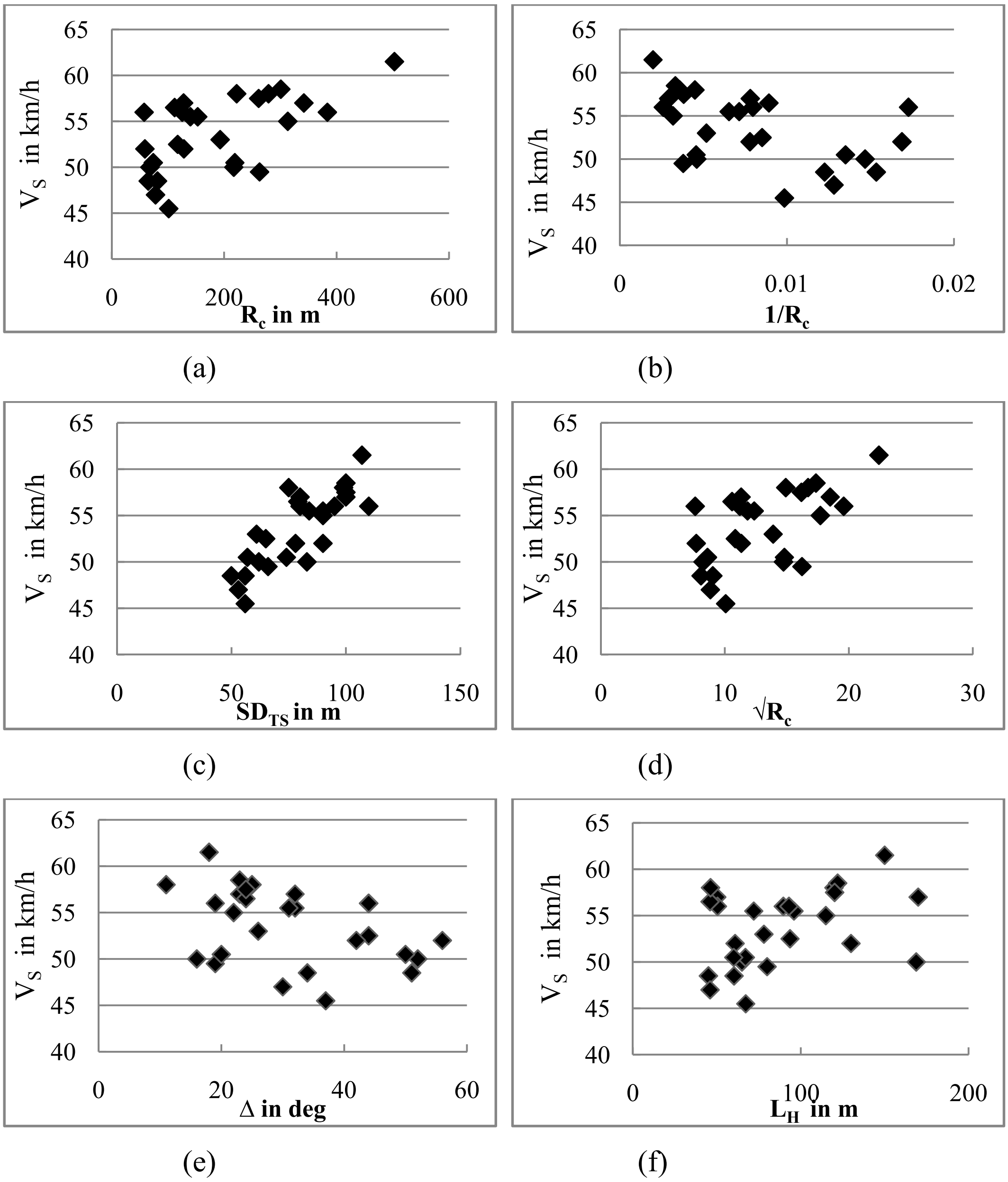


Fig.3.19 Relationship between operating speed at start point of Class B curves and geometric variables

Table 3.20 Operating speed models at start point of Class B curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_S = 57.122 - \frac{448.416}{R_c}$ (t=42.683) (t= -3.05)	0.271	0.242	9.28
2	$V_S = 38.843 + 0.186\,SD_{TS}$ (t=18.12) (t= 7.052)	0.665	0.652	49.73
3	$V_S = 39.453 + 0.004R_c + 0.169SD_{TS}$ (t=17.08) (t= 0.753) (t=4.82)	0.673	0.646	24.22
4	$V_S = 41.157 - \frac{125.558}{R_c} + 0.170SD_{TS}$ (t=13.67) (t= -1.091) (t=5.56)	0.681	0.655	25.65
5	$V_S = 38.662 + 0.126\sqrt{R_c} + 0.168SD_{TS}$ (t=17.84) (t= 0.847) (t=4.88)	0.675	0.684	24.95
6	$V_S = 71.751 + \frac{1510.927}{R_c} - \frac{354.049}{\sqrt{R_c}}$ (t=8.57) (t= 1.35) (t= -1.76)	0.355	0.301	6.6

It can be observed that all the models developed with SD_{TS} and other variables show higher regression coefficients and, because of greater influence of SD_{TS}, the coefficients of other variables are not significantly different from zero. The multi linear regression models developed, using significant variables obtained during the analysis, also failed to satisfy the model development criteria considered in the study and also the regression coefficients are less than 50% .

In this study it is aimed to develop speed prediction models of horizontal curves, using horizontal geometric features of the highway. But it is not possible to develop speed prediction models at start point of Class B curves, based on the data considered in this study. The higher influence of vehicular and driver characteristics than geometric characteristics at the start point of Class B curves may be reason for this condition. Therefore none of the models developed for this case are not considered further.

3.9.2.2 At midpoint of curve

From scatter plots shown in Fig.3.20 and correlation matrix given in Table A2.2, it is observed that the operating speed at midpoint of Class B curves shows relationship with R_c, $1/R_c$, Δ, L_H, SD_S, SD_M, $\sqrt{R_c}$, R_c^2 and $\ln(1/R_c)$.

Using these results, linear and multi linear regression analysis was performed. During the analysis it is observed that even though Δ and L_H are significant at 95% confidence level, the coefficients are not significantly different from zero. SD_S has a more logical explanation than SD_M.

Table 3.21 lists some of the models among various multi linear regression models developed. The model 2 in Table 3.21 shows a higher coefficient of regression and low PRMSE. Comparing model 1 to model 2 it is clear that V_M is more related to $\sqrt{R_c}$ than R_c. The logic of the model is that as radius and sight distance available at start point of curve increase, the operating speed at midpoint of Class B curves also increases. Increase of radius and increase of SD_S make the steering of the vehicle easy, which leads to increase of $V_{M.}$

3.9.2.3 At end point of curve

From scatter plots presented in Fig.3.21 and correlation matrix given in Table A2.2, it is observed that the operating speed at end of curve shows a good relationship with R_c, $1/R_c$, Δ, L_H, SD_S, SD_M, $\sqrt{R_c}$, R_c^2 and $\ln(1/R_c)$. Table 3.22 lists the various models developed at end point of Class B curves.All the models have a logical explanation in predicting operating speed at end point of curve and satisfy model development criteria. Comparing the model 2 in the Table 3.21 with model 4 in Table 3.22 it is observed that the variables predicting the operating speed at end point are same in the constant value in the model increased at the end point of Class B curve, which indicated the acceleration of vehicles from mid point to end point of curve.

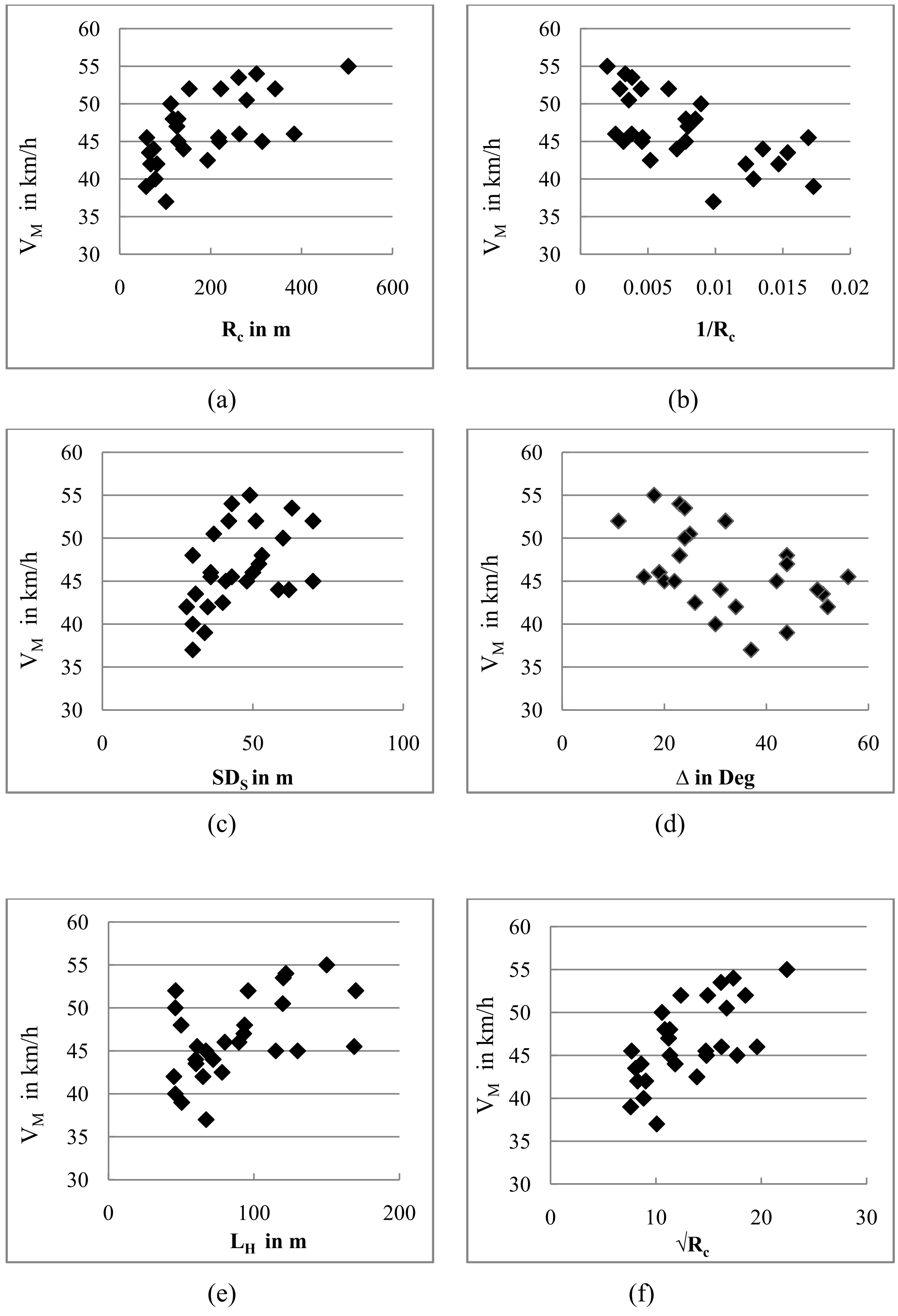


Fig.3.20 Relationship between Operating speed at midpoint of Class B curves and geometric variables

Table 3.21 Operating speed models at midpoint of Class B curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_M = 36.509 + 0.023R_c + 0.127\,SD_S$ (t=14.15) (t= 3.75) (t=2.31)	0.505	0.464	12.23
2	$V_M = 32.782 + 0.6441\sqrt{R_c} + 0.117SD_S$ (t=11.11) (t= 3.75) (t=2.1)	0.506	0.465	12.31
3	$V_M = 37.829 + 4.27x10^{-5} R_c^{\ 2} + 0.146SD_S$ (t=14.57) (t= 3.45) (t=2.62)	0.477	0.434	10.97
4	$V_M = 20.543 - 4.178\ln\left(\frac{1}{R_c}\right) + 0.108SD_S$ (t=3.71) (t= -3.66) (t=2.01)	0.498	0.456	11.89
5	$V_M = 46.128 - \frac{528.629}{R_c} + 0.098SD_S$ (t=13.07) (t= -3.23) (t=1.61)	0.455	0.41	10.03

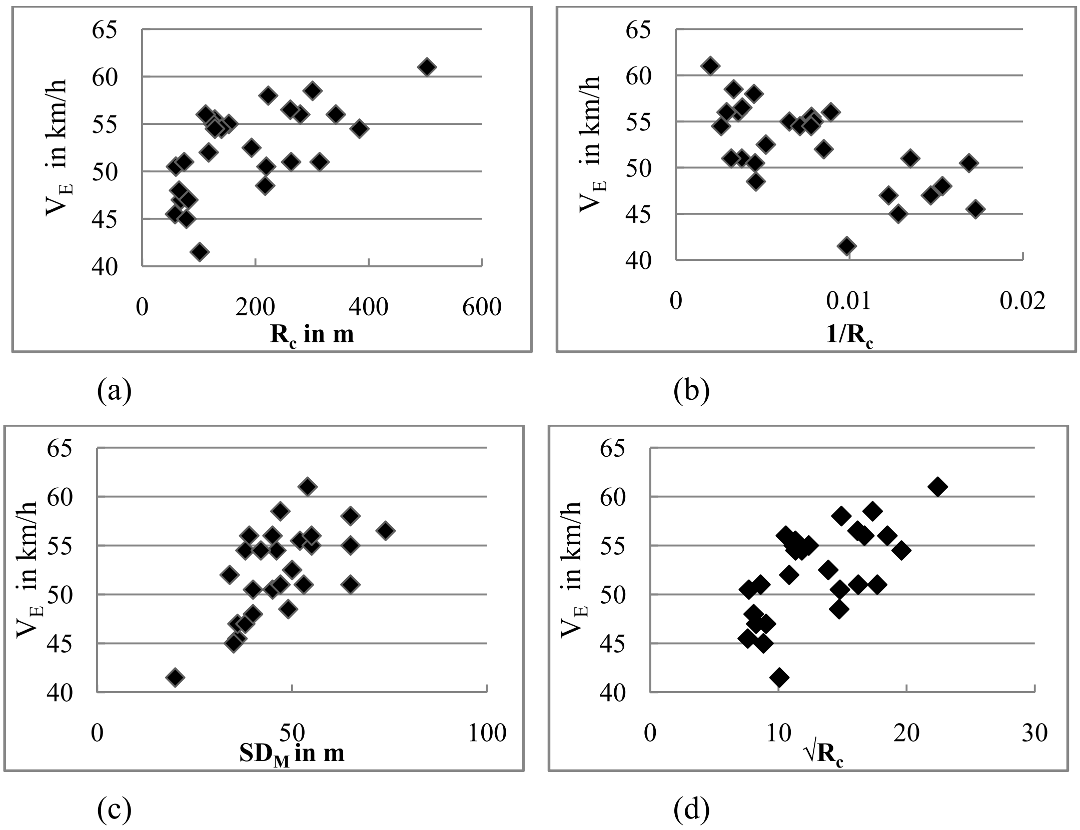


Fig.3.21 Relationship between operating speed at end point of Class B curves and geometric variables

Table 3.22 Operating speed models at end point of Class B curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_E = 42.834 + 0.729\sqrt{R_c}$ (t=18.06) (t= 4.17)	0.411	0.387	17.43
2	$V_E = 27.406 - 3.406\ln\left(\frac{1}{R_c}\right) + 0.165SD_M$ (t=5.44) (t= -3.03) (t=2.76)	0.565	0.529	15.59
3	$V_E = 48.201 - \frac{444.64}{R_c} + 0.162SD_M$ (t=12.99) (t= -2.87) (t=2.61)	0.551	0.513	14.71
4	$V_E = 37.438 + 0.52\sqrt{R_c} + 0.173SD_M$ (t=13.57) (t= 3.1) (t=2.97)	0.569	0.533	15.83
5	$V_E = 41.139 + 3.52x10^{-5}R_c^2 + 0.203SD_M$ (t=16.05) (t= 3.09) (t=3.68)	0.569	0.533	15.82

3.9.3 Models Developed for the Class C curves

Class C refers to the curves with 5.5 - 6.1 m carriageway width and 2-3 m shoulder width at middle of the horizontal curve. Out of 35 horizontal curves of class C, the data of 24 curves were used for calibration and the remaining data were used for validation. Scatter plots and correlation matrix developed were used to identify the candidate variables for predicting operating speed at study points.

3.9.3.1 At start of curve

Using the results of scatter plot Fig.3.22 and correlation matrix in Table A2.3, linear and multi linear regression analysis were performed and some of the models developed are presented in Table 3.23. Among various models developed, the coefficients of variables only in model 2 are significantly different from zero and show a higher coefficient of regression and low PRMSE. The logic of the model is that as radius increases and the coefficient of $1/R_c^2$ decreases and thereby increases of operating speed with increase of radius. But the coefficient of regression of model is less than 50 %, and hence this model is not considered further to predict the operating speed at end point of class C curves.

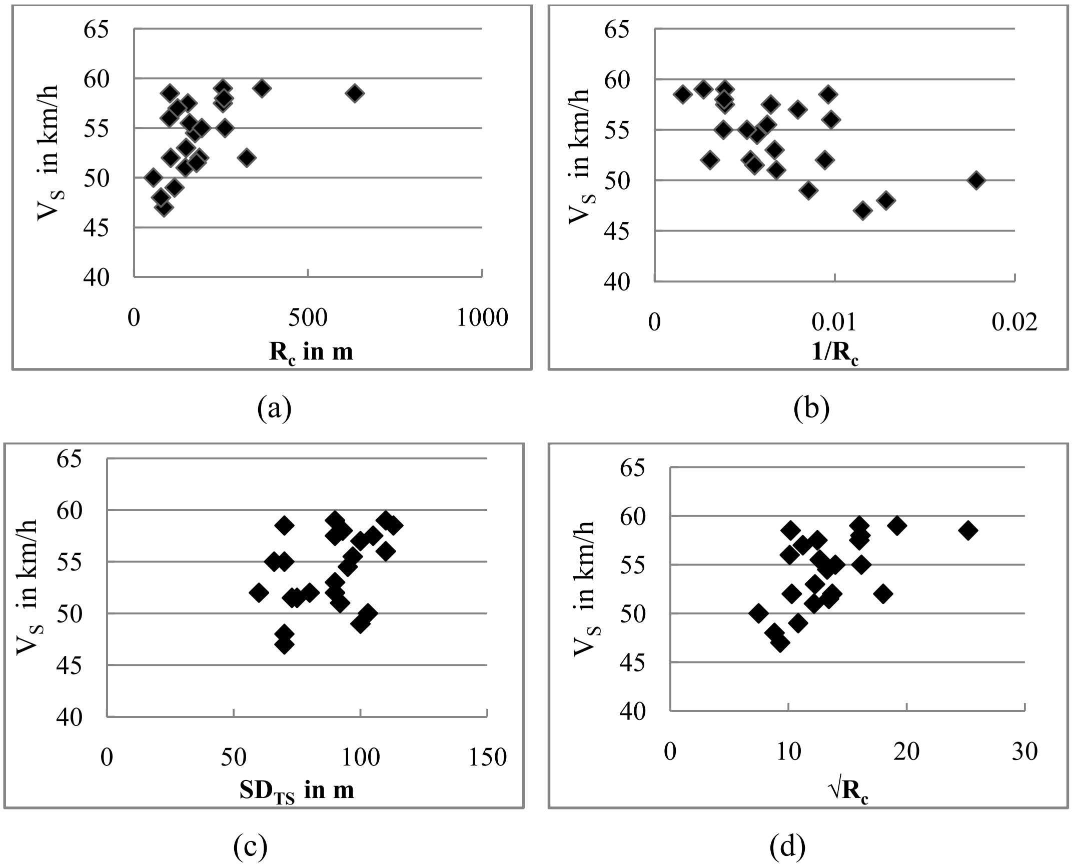


Fig3.22 Relationship between operating speed at start point of Class C curves and geometric variables

3.9.3.2 At midpoint of curve

From the Fig. 3.23 and correlation matrix in Table A2.3 it is observed that R_c, $1/R_c$, Δ, L_H, SD_S, $1/\sqrt{R_c}$,$1/R_c^2$, and $\ln(1/R_c)$ are the variables having an influence on predicting operating speed at midpoint of class C curves. Table 3.24 presents the regression models developed, which have higher regression coefficients. Among various models developed, model 4 shows a higher coefficient of regression and low PRMSE, and therefore, is recommended to predict operating speed at midpoint of class C curves.

The model logic is that as as radius increases the value of $1/\sqrt{R_c}$ decreases, thereby results in increase of operating speed at mid point of curve. Model 5 shows a higher coefficient of regression, but fails to explain the model's logic.

Table 3.23 Operating speed models at start point of Class C curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_S = 51.228 - \frac{502.109}{R_c} + 0.076SD_{TS}$ (t=12.64) (t= -2.98) (t=1.86)	0.466	0.403	7.42
2	$V_S = 48.088 - \frac{26650.6}{R_c^2} + 0.092SD_{TS}$ (t=13.13) (t= -3.05) (t=2.27)	0.475	0.413	7.68
3	$V_S = 55.181 - \frac{84.346}{\sqrt{R_c}} + 0.069SD_{TS}$ (t=11.02) (t= -2.79) (t=1.61)	0.442	0.377	6.74
4	$V_S = 32.974 + 3.111\ln\left(\frac{1}{R_c}\right) + 0.063SD_{TS}$ (t=5.16) (t= 2.49) (t=1.39)	0.404	0.334	5.773

3.9.3.3 At end point of curve

From the scatter plot in Fig.3.24 and correlation matrix in Table A2.3 it is clear that none of the variables considered in this study shows a significant relationship to predict operating speed at end point of class C curves, hence, it is not possible to develop operating speed model at the end of curve. The reason may be that as shoulder width available at mid of the curve increases, driver reaction or behaviour may change due to increase in sight distance available, which results in reduced effect of geometric variables considered in the study.

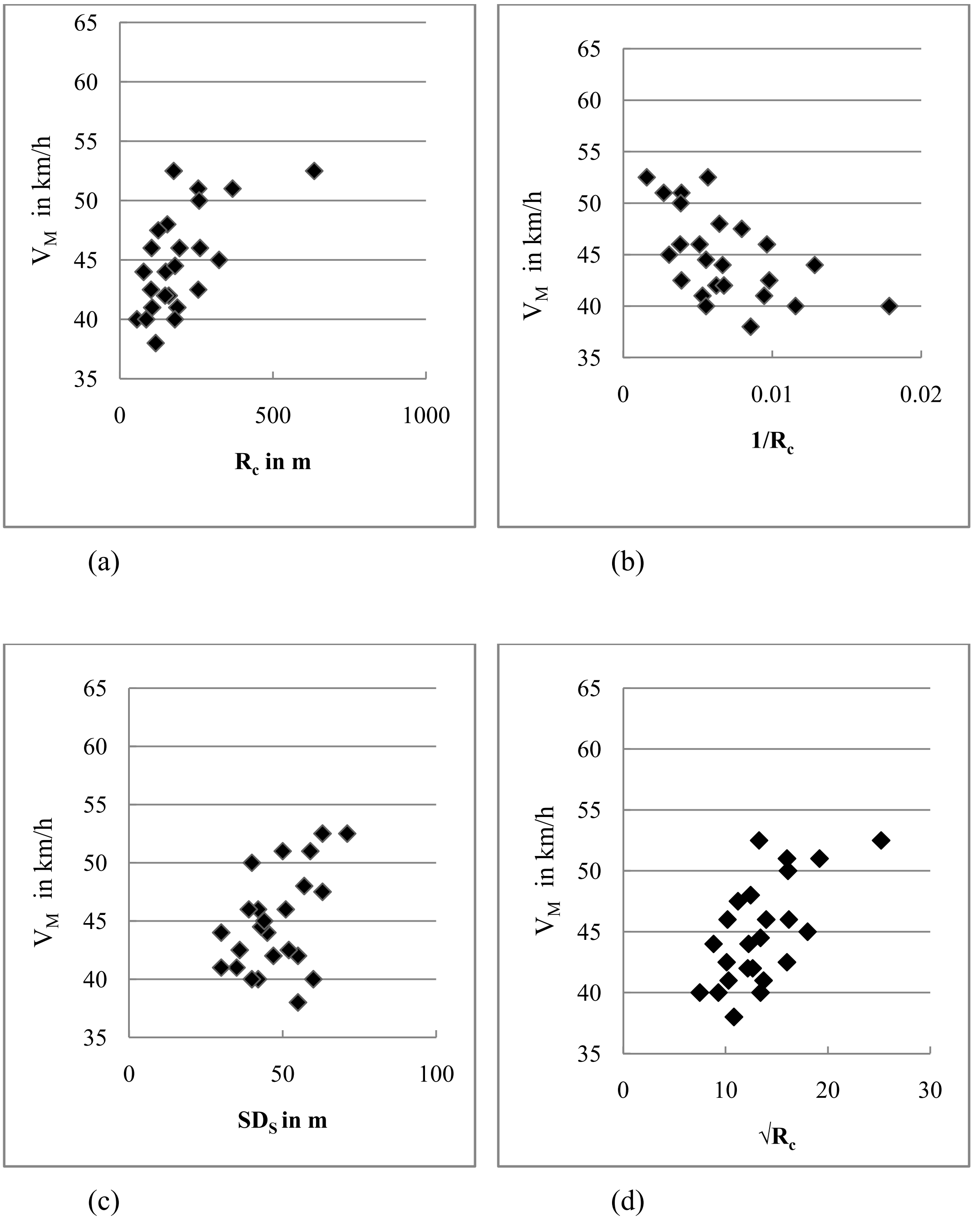


Fig3.23 Relationship between Operating speed at midpoint of Class C curves and geometric variables

Table 3.24 Operating speed models at midpoint of Class C curves

Model No.	Models (t-values)	R^2	R_a^2	F-value
1	$V_M = 34.471 + 0.239\,SD_S$ (t=11.09) (t= 3.73)	0.436	0.405	13.92
2	$V_M = 42.028 - \frac{489.99}{R_c} + 0.152\,SD_S$ (t=10.96) (t= -2.74) (t=2.39)	0.609	0.563	13.22
3	$V_M = 38.756 - \frac{21273.3}{R_c^2} + 0.177\,SD_S$ (t=11.31) (t= -2.20) (t=2.75)	0.561	0.510	10.88
4	$V_M = 46.357 - \frac{90.784}{\sqrt{R_c}} + 0.143\,SD_S$ (t=9.76) (t= -2.98) (t=2.29)	0.63	0.587	14.48
5	$V_M = 20.594 + 3.674\ln\left(\frac{1}{R_c}\right) + 0.141 SD_S$ (t=3.97) (t= -3.07) (t=2.28)	0.638	0.595	14.97

3.9.4 Recommended Models

In this study several models are developed for the evaluation of horizontal curves in intermediate lane rural highways. All the selected models have a logical explanation in predicting operating speed at study points. These models are used to evaluate the consistency of horizontal curves of existing intermediate lane rural highways. Even though the intention of this study to find out the effect of geometric variables at start, mid, and end points of horizontal curve in some classes, it was not possible to predict operating speed at start and end points of curve, the reason being that the influence of other characteristics is more than the geometric characteristics.

Most drivers travel as fast as they feel comfortable on a straight section and slow down only where necessary (Hassen et al. 2000). Maximum difference in operating speeds on two successive sections occurs between tangent and midpoint of the curve (Lamm et al.1988, Gibreel et al. 1999, Fitzpatrick et al. 2000a). Therefore, among the various models, operating speeds at tangent point and midpoints of classified curves A, B, and C are recommended for consistency evaluation of horizontal curves. The recommended models with its PRMSE are presented in the Table 3.25.

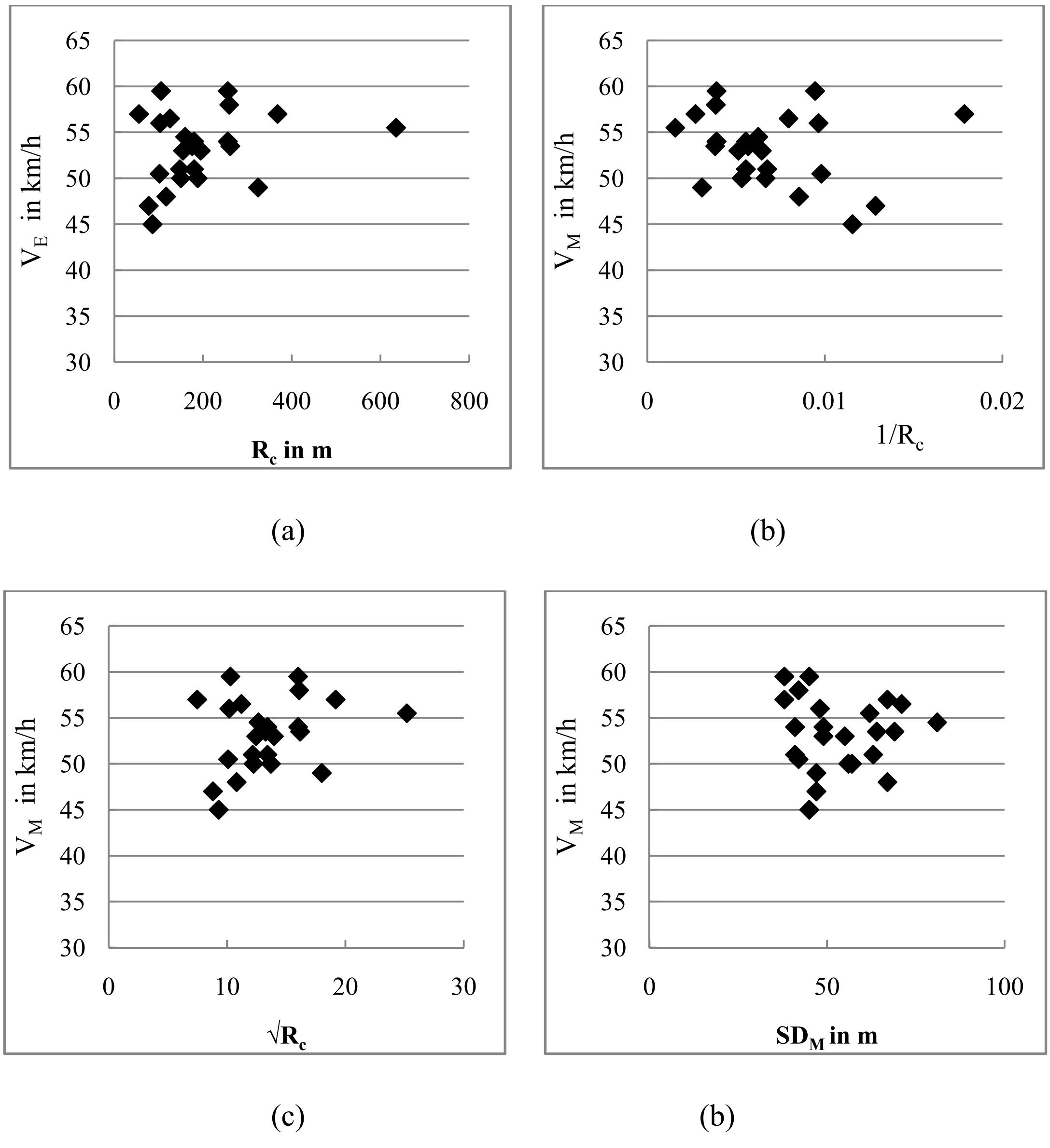


Fig.3.24 Relationship between operating speed at end point of Class C curve and geometric variables

Table 3.25 Recommended Operating speed models for Intermediate lane rural highways

Class	Point at curve	Model	R^2	PRMSE	
				Calibration	Validation
	Tangent	$V_{TM} = 56.108 + 0.139\,SD_{TS} - \dfrac{54.731}{\sqrt{PTL}}$	0.590	4.2	6.4
A	Midpoint	$V_M = 17.383 - 5.109\ln\left(\dfrac{1}{R_c}\right) + 0.098SD_S$	0.557	7.16	7.9
B	Midpoint	$V_M = 32.782 + 0.6441\sqrt{R_c} + 0.117SD_S$	0.506	7.0	9.7
C	Midpoint	$V_M = 46.357 - \dfrac{90.784}{\sqrt{R_c}} + 0.143\,SD_S$	0.63	7.9	9.8

3.9.5 Validation of Recommended Models

Fig.3.25 and Figs.3.26 (a) to (c), show the comparisons of calibration and validation data sets for four recommended models, which are presented in Table 3.25.

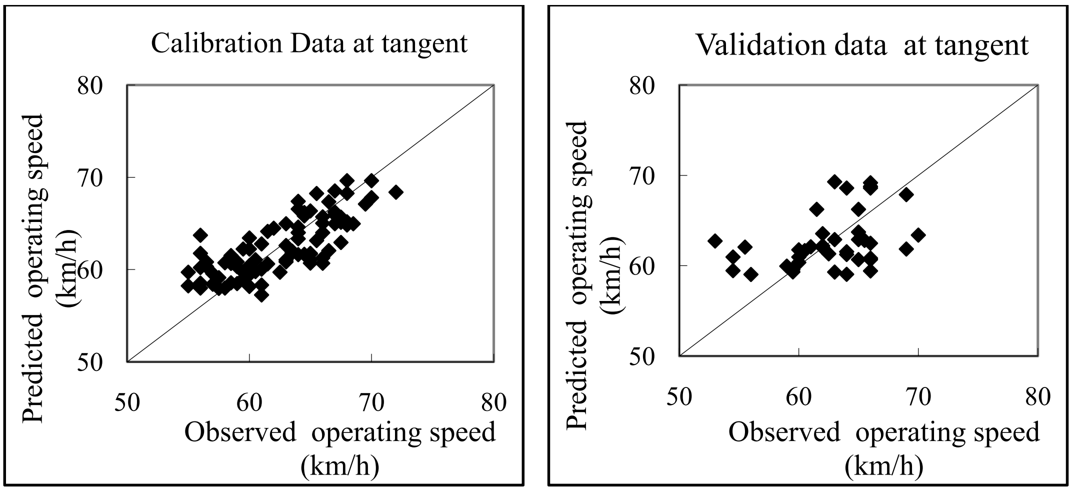
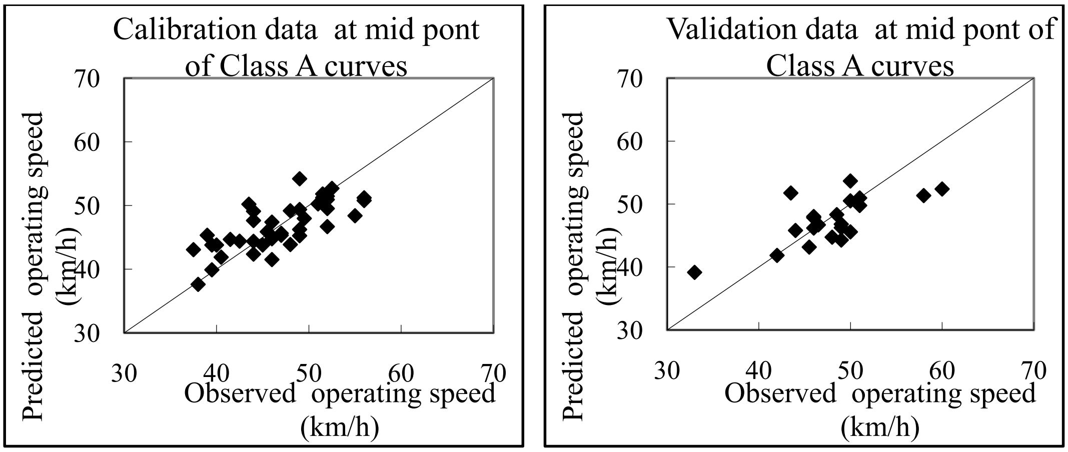
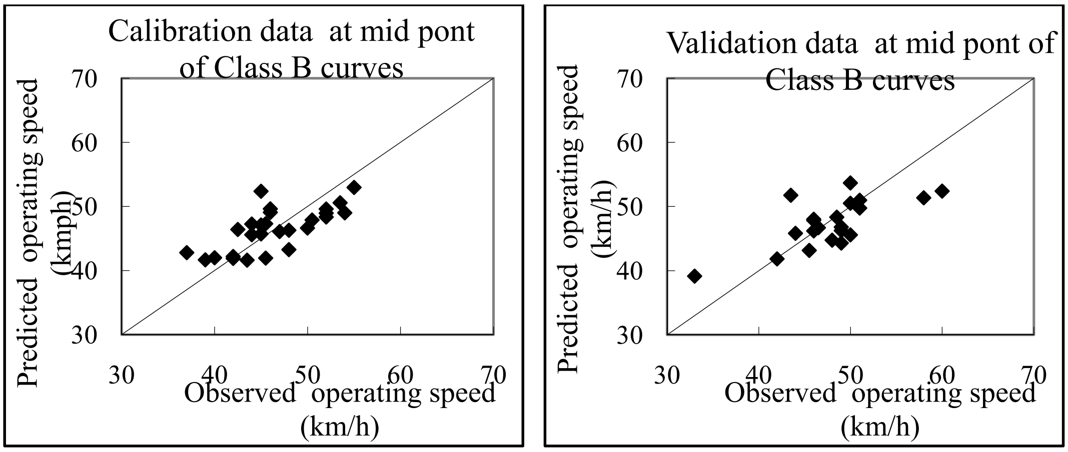


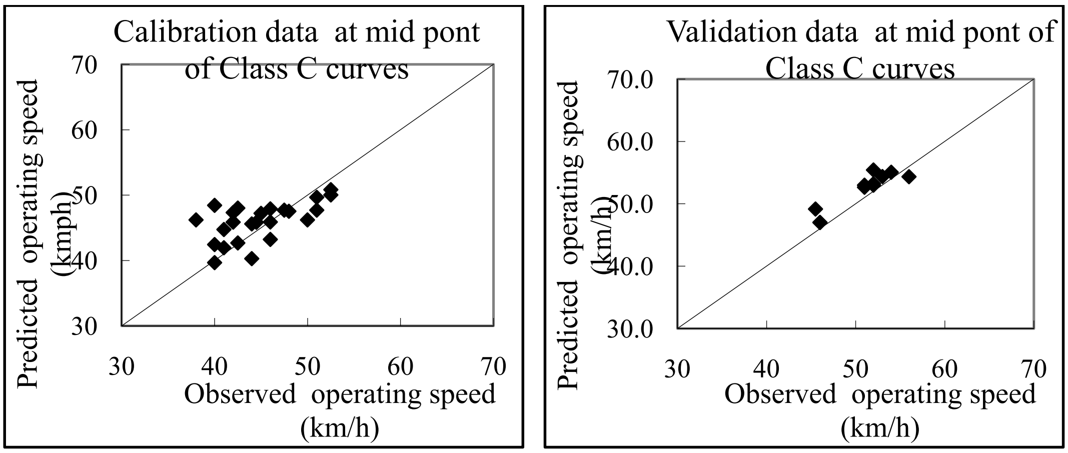
Fig. 3.25 Comparison between observed and predicted operating speed values at tangent point of curves in Intermediate lane rural highways



(a) At midpoint of Class A curves



(b) **At midpoint of Class B curves**



(c) **At midpoint of Class C curves**

Fig. 3.26 Comparison between observed and predicted operating speed values of classified curves

Comparisons were made between the observed and predicted values of 85th percentile speed for calibration and validation data separately. Scatter plots of observed and predicted values were prepared to check whether the points lie close to 45^0 line or not. It can be seen that the points lie very close to the 45º line and hence it can be said that operating speed models at all the study points are satisfactory in predicting the operating speed. For both data sets, the percentage Root Mean Square Error (PRMSE) values are calculated to check the goodness of fit of the models. The low PRMSE values (Table 3.25), indicate that the operating speed models are good in predicting the operating speed at all the three study points

3.10 DEVELOPMENT OF SPEED DIFFERENTIAL MODELS

As the speed distributions at the tangent and curve sections need not be the same (Hirsh 1987 and McFadden and Elefteriadou 2000) and even if the speed distributions are the same, the 85^{th} percentile drivers need not be the same at two locations (Misaghi and Hassan 2005). Hence, it is suggested that the full distribution of speed changes as incurred by each driver should be examined to calculate the speed differential value. In this study, along with operating speed models, 85^{th} percentile speed differential models were also developed for intermediate lane rural highways.

3.10.1 Prediction of 85^{th} Speed Differential Models

One of the agreed facts is that most drivers reduce their speed while approaching the curve and that this speed is minimum at the middle of curve (Lamm et al. 1995, Fitzpatrick et al. 2000a). Therefore, maximum reduction in speed takes place between tangent and midpoint of curve (Al-Masaeid et al. 1995, McFadden and Elefteriadou 2000) and this may cause unusual accident situations (Collins et al. 1999). Hence in this study, speed differential between tangent point and middle of the horizontal curve ($(\Delta V)_{85TMM}$) is considered for model development.

To get uniformity among the various geometric data collected, initially the curves having 5.5-6.1m carriageway width were considered for the study. Graphical (Fig.3.27) and correlation analysis (Table 3.26) were conducted to identify the predictor variables that are significant at 5% level of significance. It is clear from both analyses that radius, inverse of radius, deflection angle, ratio of preceding tangent length to radius, ratio of radius to preceding tangent length, and shoulder width available at curve are the influencing variables to predict speed differential between tangent and curve which satisfy at 95% confidence level. It can be observed from Fig.3.27 (a) that the radius of the curve has a great effect on the speed differential value for the radii of less than approximately 250 m, but this effect tends to vanish after the 400 m radius. It is clear from Fig.3.27 (c) that PTL does not have any influence on predicting speed differential, indicates that PTL alone does not cause any reduction in speed.

Having identified significant variables several trials were performed to develop linear and multi linear regression models and are tabulated in Table 3.27. In this study, it should be emphasised that although the models have a relatively low coefficient of regression, the variables in each model are statistically significant at 95% level of confidence level. It is observed in Table 3.27 that coefficient of all the variables used in the model are different from zero (satisfying t-test).

From models 1 and 2, it is clear that the inverse of radius having a higher coefficient of regression indicates that variable $1/R_c$ has a higher influence than R to predict $(\Delta V)_{85TMM}$. The negative sign of R_c indicates that as radius increases, reduction in speed from tangent to curve decreases, while positive sign of $1/R_c$ indicates that as radius decreases, the value of $1/R_c$ increases, hence, reduction in speed is more. In model 3 the deflection angle of the curve also shows effect on speed differential, greater deflection angle results due to sharper curve, causing higher speed reduction on curve. This model shows similar findings of Misaghi and Hassan (2005). In the model 4, the positive sign of ratio PTL/R_c indicates that as this ratio increases, $(\Delta V)_{85TMM}$ also increases. But this increase in speed differential depends on whether both PTL and R_c increase and only R_c decreases or increases. If a long straight stretch follows a sharp curve, it causes higher speed differential from tangent to curve. The positive sign of shoulder width S_M indicates that as this value increases $(\Delta V)_{85TMM}$ also increases. Actually, increase in shoulder width increases the sight distance available to the driver, hence the speed differential decreases. Therefore the model 6, 7, 11 and 12 will show a logical explanation, only when an increase in shoulder width is followed by sharper and longer curves.

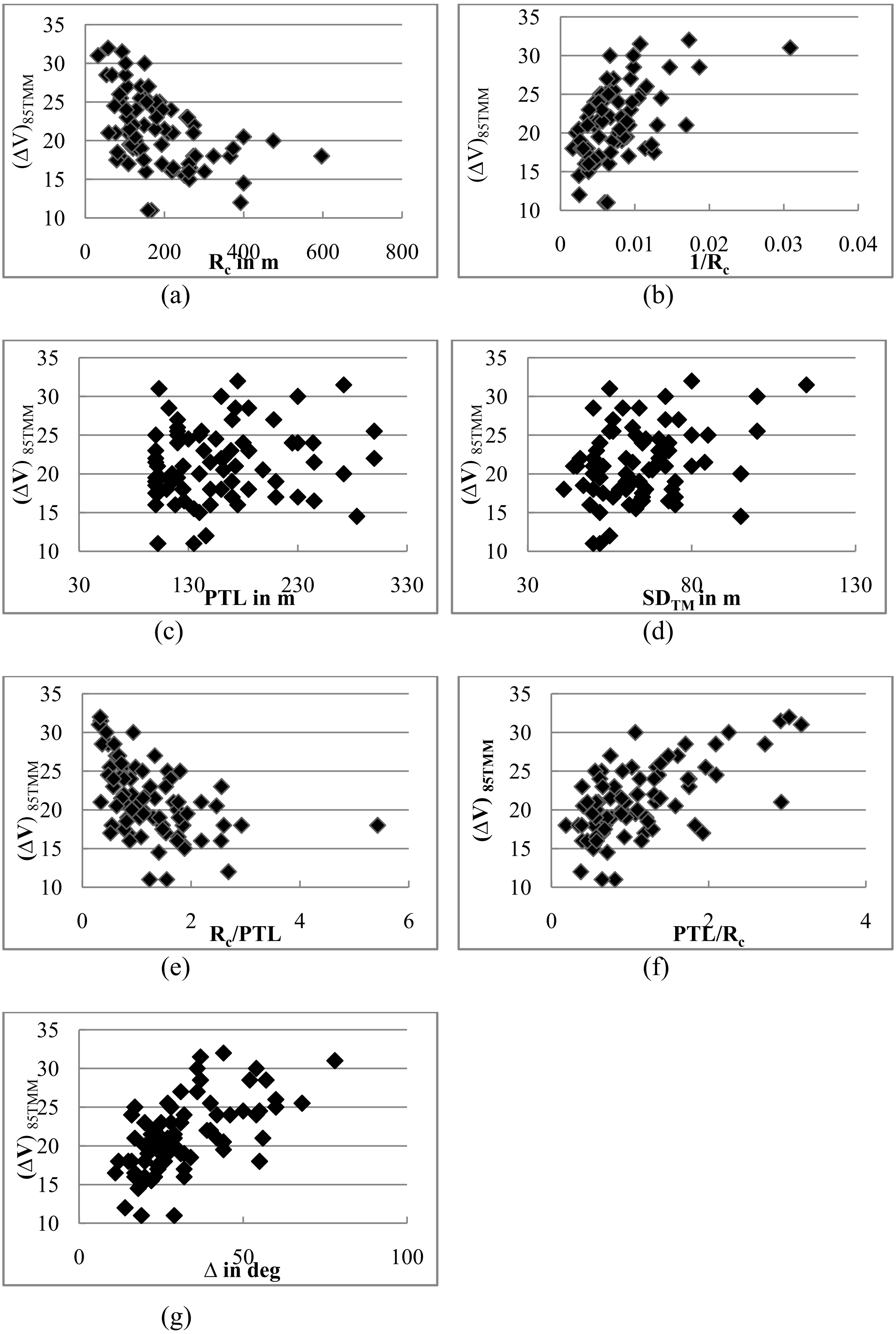


Fig.3.27 Relationship between speed differential and significant variables

Table 3.26 Correlation Matrix for 85^{th} Speed differential

	R_c	$1/R_c$	Δ	L_H	PTL	$(\Delta V)_{85TMM}$	PTL/R_c	R_c/PTL	e_S	e_M	e_E	SD_{TS}	SD_{TM}	SD_S	SD_M	Ss	S_M	S_E
R_c	1																	
$1/R_c$	-0.75	1																
Δ	-0.63	0.71	1															
L_H	0.47	-0.48	0.12	1														
PTL	0.17	-0.17	0.05	0.39	1													
$(\Delta V)_{85TMM}$	-0.47**	0.52**	0.60**	0.03	0.16	1												
PTL/R_c	-0.67	0.82	0.71	-0.28	0.34	0.62**	1											
R_c/PTL	0.84	-0.63	-0.60	0.25	-0.33	-0.47**	-0.75	1										
e_S	-0.01	-0.05	-0.15	-0.17	-0.01	-0.08	-0.05	-0.02	1									
e_M	-0.42	0.30	0.34	-0.16	-0.18	0.09	0.18	-0.30	0.10	1								
e_E	-0.18	0.01	0.11	0.00	0.06	0.09	0.07	-0.16	0.05	0.38	1							
SD_{TS}	0.27	-0.27	-0.01	0.46	0.58	0.19	0.04	0.00	-0.08	-0.09	0.20	1						
SD_{TM}	0.19	-0.15	0.07	0.41	0.49	0.32**	0.14	-0.01	-0.13	-0.05	0.14	0.71	1					
SD_S	0.02	-0.21	0.06	0.26	0.31	-0.06	-0.02	-0.08	0.00	0.10	0.00	0.44	0.35	1				
SD_M	0.31	-0.38	-0.19	0.27	0.19	-0.13	-0.27	0.22	-0.03	0.03	0.11	0.49	0.43	0.53	1			
S_S	-0.12	0.01	0.04	0.08	-0.04	0.31**	0.07	-0.05	0.21	-0.11	-0.01	0.17	0.14	0.19	0.13	1		
S_M	-0.09	-0.05	0.09	0.19	-0.07	0.33**	0.00	0.00	0.25	-0.05	-0.08	0.17	0.16	0.24	0.15	0.83	1	
S_E	-0.06	-0.12	0.01	0.15	-0.03	0.26*	-0.05	0.00	0.28	-0.08	-0.06	0.09	0.07	0.16	0.04	0.79	0.89	1

Where.** Correlation is significant at the 0.01 level (2-tailed) and * Correlation is significant at the 0.05level (2-tailed).

Table 3.27 Speed differential models for the intermediate lane rural highways

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)							R^2	Ra^2	F-value
		R_c	$1/R_c$	Δ	PTL/R_c	SD_{TM}	S_S	S_M			
1	25.050 (26.626)	-0.21 (-4.612)		-	-	-	-		0.216	0.206	21.27
2	17.299 (20.013)		537.94 (5.38)		-	-	-		0.273	0.264	28.97
3	14.939 (14.18)			0.198 (6.55)					0.358	0.350	42.96
4	16.556 (20.717)				4.235 (6.903)				0.382	0.374	47.65
5	14.428 (6.09)					0.106 (2.96)			0.102	0.090	8.75
6	19.511 (24.357)						1.606 (2.826)		0.094	0.082	7.98
7	19.12 (22.277)							1.859 (3.081)	0.110	0.098	9.49
8	14.882 (14.92)			0.107 (2.64)	2.673 (3.19)				0.434	0.419	29.17
9	9.786 (4.99)			0.111 (2.89)	2.369 (2.95)	0.082 (2.97)			0.494	0.474	24.39
10	11.734 (6.07)				4.006 (6.7)	0.079 (2.73)			0.437	0.423	29.55
11	11.146 (6.0)				3.924 (6.88)	0.068 (2.4)	1.24 (2.8)		0.492	0.472	24.23
12	10.738 (5.95)				4.050 (7.36)	0.063 (2.32)		1.69 (3.74)	0.526	0.501	27.73

Among several multi linear regression models developed, the model 9 in Table 3.27 has a higher R^2 value and a better logical explanation. But this coefficient of regression (R^2) is less than 50%. During the analysis, it is observed that shoulder width available at curve has a significant effect on speed differential. Hence, to predict better models the curves in the data set are classified further based on shoulder width available at the middle of the curve.

McFadden and Elefleriadou (2000) and Misaghi and Hassan (2005) found that the simple subtraction of operating speeds at the approach tangent and middle of the curve underestimates the real value of speed differential. To check this, the scatter plot of observed 85th speed differential, $(\Delta V)_{85TMM}$ and reduction calculated using operating speed between tangent and curve (ΔV_{85}) was prepared. Fig.3.28 shows the relationship between $(\Delta V)_{85TMM}$ and ΔV_{85} and the best fit line indicates that $(\Delta V)_{85TMM}$ is 9.82 km/h greater than ΔV_{85} in intermediate lane highways.

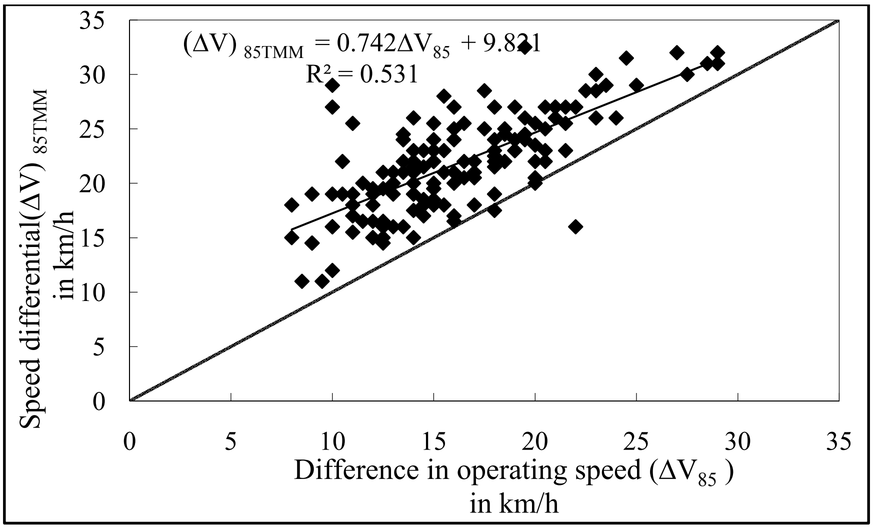


Fig.3.28 Relation between $(\Delta V)_{85TMM}$ and ΔV_{85}

This confirms the conclusion of the earlier studies (McFadden and Elefleriadou 2000, Misaghi and Hassan 2005, and Park and Saccomanno 2006) that calculation of speed differential by the simple subtraction of the two related 85thpercentile speed values on tangent and curve (ΔV_{85}) underestimates the 85th percentile speed differential $(\Delta V)_{85TMM.}$

3.11 SPEED DIFFERENTIAL MODELS DEVELOPED BASED ON CLASSIFICATION

The speed reduction is expected to increase with narrow shoulders (Misaghi and Hassan 2005) and correlation exists between speed reduction into the curve and length of approach tangent, pavement width, shoulder width (McFadden and Elefteriadou 2000). The maximum speed reduction takes place between tangent and curve (Al-Masaeid et al. 1995). Hence the horizontal curves collected are classified based on the shoulder width available at the middle of curve. The three classes of curves are: Class A curves with 0 to 1m shoulder width, Class B curves with 1 to 2 m shoulder width and Class C curves with 2 to 3 m shoulder width. Modelling was done separately for these classes of horizontal curves. In each class 2/3rd of data was used for calibration and 1/3rd of data was used for validation.

3.11.1 Speed Differential Models for Class A Curves

In this case, out of 65 curves, data of 44 curves were used for the development of speed differential models. From scatter plots, shown in Figs.3.29 (a) to (f), and correlation matrix, given in Table 3.28, it is observed that speed differential between tangent and mid of curve shows a significant relationship with radius, deflection angle, inverse of radius, $\sqrt{R_c}$ and $1/\sqrt{R_c}$.

Several trials were performed to get the most logical explanatory models using linear regression technique. The model in Table 3.29 is found to be better in terms of goodness of fit statistics and logic and shows the highest R^2 value. The reduction in speed increases as PTL/R_c increases. Several trials were also conducted to develop multi linear regression models using significant variables, which showed higher coefficient of regression, but were found to have t-test value of less than two. Hence the models 8 to 13 presented in Table 3.29 are not considered further.

3.11.2 Speed Differential Models for Class B Curves

Scatter plots shown in Fig.3.30 and correlation matrix presented in Appendix 2, Table A 2.4, were used to identify predicting speed differential variables.

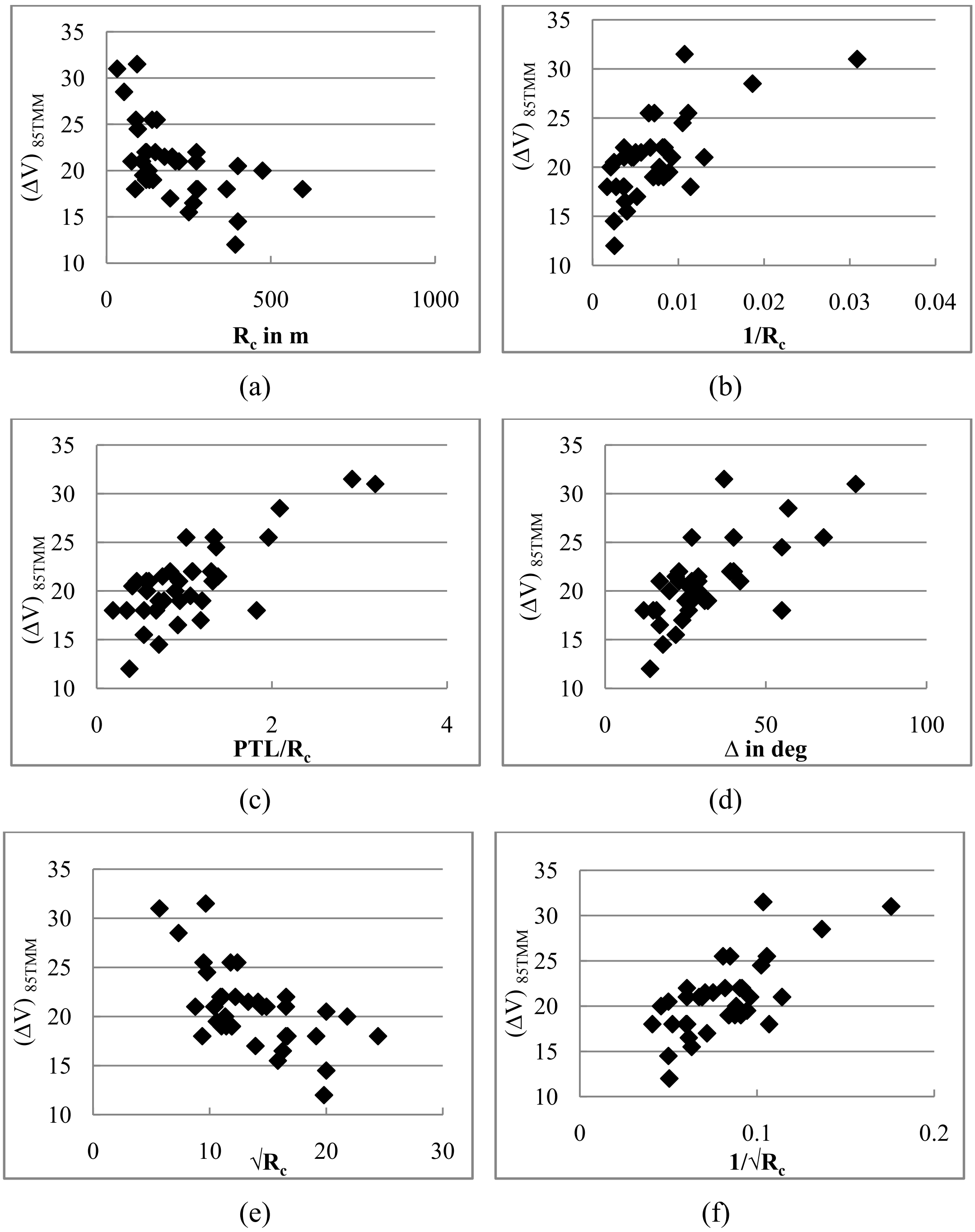


Fig.3.29 Relationship between speed differential and significant variables for the Class A curves

Table 3.28 Correlation matrix developed for the Class A curves

	R_c	$1/R_c$	Δ	L_H	PTL	ΔV_{85TMM}	PTL/ R_c	R_c /PTL	SD_{TS}	SD_{TM}	SD_S	SD_M	$\sqrt{R_c}$	R_c^2	$1/\sqrt{R_c}$	$1/R_c^2$	PTL/$\sqrt{R_c}$
R_c	1																
$1/R_c$	-0.70	1															
Δ	-0.67	0.80	1														
L_H	0.59	-0.54	-0.07	1													
PTL	0.28	-0.33	-0.08	0.56	1												
$(\Delta V)_{85TMM}$	-0.56	0.68	0.69	-0.21	-0.01	1											
PTL/ R_c	-0.66	0.81	0.80	-0.30	0.19	0.77	1										
R_c /PTL	0.84	-0.55	-0.60	0.30	-0.22	-0.49	-0.69	1									
SD_{TS}	0.36	-0.36	-0.10	0.63	0.72	0.05	-0.02	0.03	1								
SD_{TM}	0.28	-0.20	-0.02	0.47	0.58	0.21	0.17	0.05	0.71	1							
SD_S	-0.01	-0.15	0.16	0.33	0.56	-0.05	0.19	-0.19	0.52	0.42	1						
SD_M	0.35	-0.35	-0.21	0.37	0.32	-0.17	-0.19	0.25	0.61	0.51	0.46	1					
$\sqrt{R_c}$	0.99	-0.80	-0.73	0.62	0.31	-0.62	-0.73	0.80	0.39	0.27	0.02	0.37	1				
R_c^2	0.96	-0.54	-0.55	0.52	0.19	-0.44	-0.54	0.87	0.29	0.28	-0.04	0.31	0.90	1			
$1/\sqrt{R_c}$	-0.83	0.97	0.81	-0.60	-0.35	0.70	0.82	-0.65	-0.40	-0.22	-0.10	-0.38	-0.91	-0.69	1		
$1/R_c^2$	-0.45	0.93	0.69	-0.41	-0.25	0.59	0.72	-0.35	-0.25	-0.14	-0.20	-0.27	-0.56	-0.31	0.83	1	
PTL/$\sqrt{R_c}$	-0.42	0.38	0.54	0.06	0.67	0.56	0.83	-0.68	0.36	0.42	0.49	0.03	-0.43	-0.39	0.41	0.29	1

Table 3.29 Speed differential models for the intermediate lane rural highways of Class A curves

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)								R^2	Ra^2	F-value
		R_c	$1/R_c$	Δ	PTL/R_c	$PTL/\sqrt{R_c}$	$\sqrt{R_c}$	$1/\sqrt{R_c}$	$1/R_c^2$			
1	24.476 (22.29)	-0.018 (-3.9)		-	-	-	-			0.312	0.291	15.39
2	17.050 (19.85)		519.522 (5.447)		-	-	-			0.466	0.45	29.67
3	14.955 (12.75)			0.187 (5.54)						0.475	0.459	30.73
4	15.746 (18.45)				4.797 (6.986)					0.589	0.577	48.79
5	14.937 (9.35)					0.473 (3.95)				0.315	0.295	15.61
6	12.27 (7.685)							105.759 (5.64)		0.483	0.468	31.82
7	28.991 (15.573)						-0.599 (-4.545)			0.383	0.365	21.118
8	15.723 (18.36)		130.693 (0.906)		3.924 (3.31)					0.599	0.575	24.68
9	15.063 (14.59)			0.057 (1.16)	3.746 (3.30)					0.606	0.582	25.33
10	16.742 (9.164)	-0.003 (-0.618)			4.42 (4.792)					0.594	0.569	24.146
11	14.371 (9.148)				3.771 (3.143)			30.438 (1.04)		0.602	0.578	25.0
12	15.953 (16.48)				4.456 (4.45)				1882.6 (0.473)	0.592	0.567	23.95
13	18.089 (5.99)				4.206 (4.19)		-0.126 (-0.816)			0.597	0.573	24.48

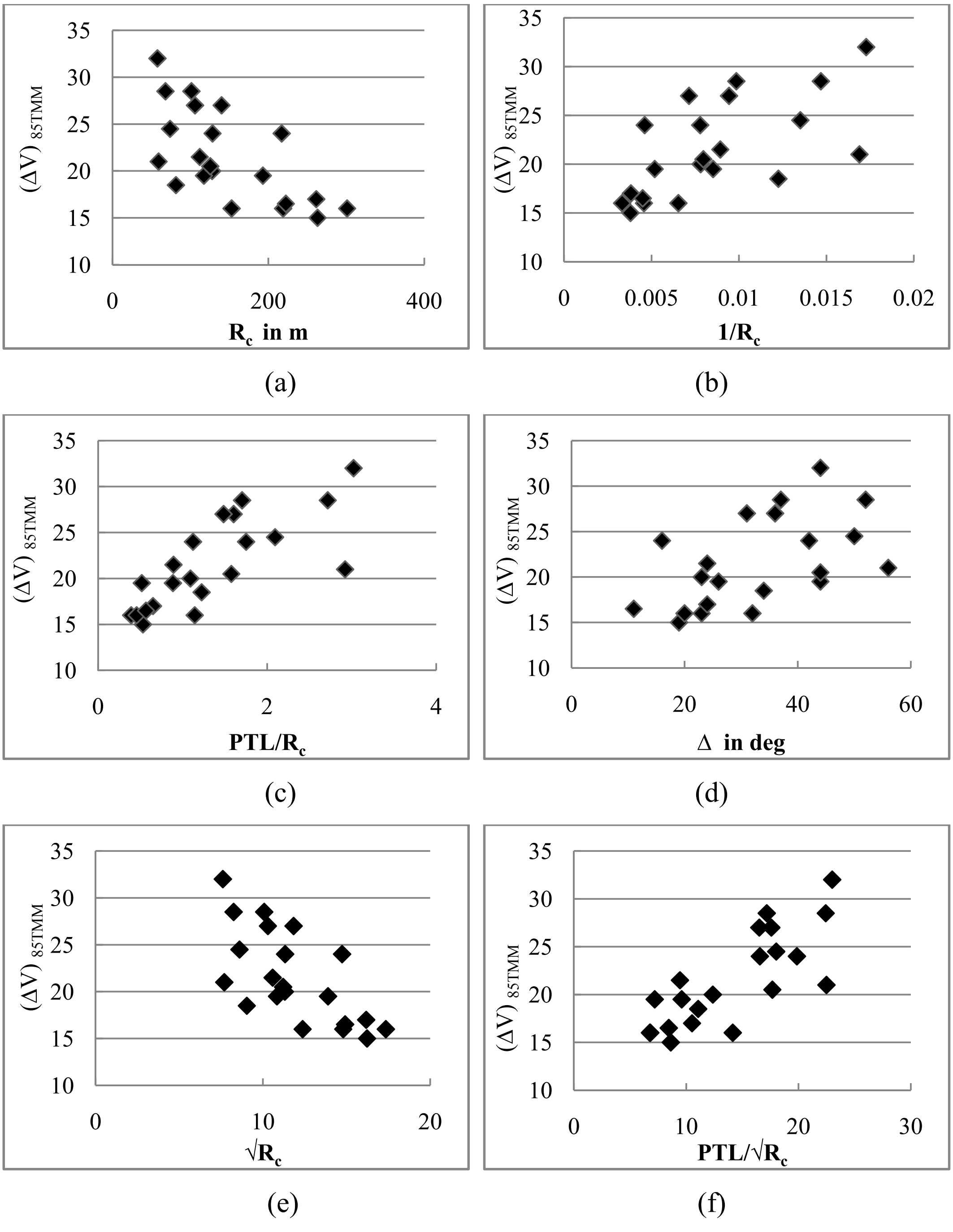


Fig.3.30 Relationship between speed differential and significant variables for the Class B Curves

The variables which are significant at 95% confidence level were used in stepwise regression analysis to predict the speed differential at tangent to midpoint of the curve. Different models were developed, for the Class B are presented in the Table 3.30.

The model 5 with $PTL/\sqrt{R_c}$ is better than other models, and hence is selected for the consistency evaluation. The positive sign of $PTL/\sqrt{R_c}$ shows an increased speed differential with increase in tangent length and decrease in radius of curve. As the curve becomes sharper, a driver will find it extremely difficult to negotiate the curve. Consequently, the driver will reduce the speed.

On comparing the model 5 in Table 3.29 with model 5 in Table 3.30 it can be observed that the predicting variables are the same in both the models and the constant value in the model decreases as shoulder width increases at the midpoint of the curve. This indicates that increase of shoulder width available at midpoint of the curve decreases the speed differential at that point. Increase in shoulder width increases the side clearance of road, which increases the visibility ahead of the driver; therefore, drivers would not reduce their speed suddenly.

3.11.3 Speed differential models for class C curves

Fig.3.31 and correlation matrix in Table A2.5 shows the relationship between significant variables and speed differential between tangent and midpoint of Class C curves. It can be observed from that R_c, R_c/ PTL, $\sqrt{R_c}$ and R_c^2 are the more influencing variables than others. Having identified significant variables several trials were performed to develop linear and multi linear regression models for the class C curves, which are tabulated in Table 3.31.

It is observed from Table 3.31 that although the models have a relatively low coefficient of regression, the variables in each model are statistically significant at 95% level of confidence. Among different models developed, none of the models has a coefficient of regression more than 50 %. Lower coefficient of regression explains that contribution of selected geometric variables is not significant to the estimation of speeds or vehicular and driver characteristics that may have the influence on speed differentials. Therefore, one can be conclude that larger the shoulder width available at middle of curve, influence of geometric variables is less. Hence it is not possible to recommend the model to predict the speed differentials of Class C curves.

Table 3.30 Speed differential models for the intermediate lane rural highways of Class B curves

Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)									R^2	Ra^2	F-value
		R_c	$1/ R_c$	Δ	PTL/ R_c	R_c /PTL	$PTL/\sqrt{R_c}$	$\sqrt{R_c}$	$1/\sqrt{R_c}$	$1/ R_c^{2}$			
1	28.291 (14.819)	-0.045 (-3.915)		-	-	-	-				0.415	0.384	13.453
2	15.202 (7.891)		747.042 (3.666)								0.447	0.417	15.33
3	15.215 (10.64)				4.689 (5.12)						0.58	0.558	26.23
4	27.444 (19.19)					-5.577 (-4.82)					0.55	0.527	23.28
5	11.522 (5.948)						0.711 (5.533)				0.617	0.597	30.62
6	34.936 (10.096)							-1.129 (-3.98)			0.455	0.426	15.83
7	8.561 (2.462)								145.11 (3.846)		0.438	0.408	14.79
8	15.91 (9.56)		-356.03 (-0.836)		6.414 (2.83)						0.596	0.551	13.25
9	17.775 (3.901)	-0.010 (-0.593)			3.931 (2.48)						0.588	0.542	12.84
10	18.040 (2.352)				4.142 (2.305)			-0.176 (0.368)			0.583	0.537	12.59
11	16.836 (3.7)				5.43 (2.5)				-29.32 (-0.38)		0.583	0.537	12.59
12	13.8 (8.56)				7.863 (3.74)					-32111 (-1.66)	0.636	0.595	15.71
13	9.844 (3.358)				0.598 (3.05)				36.563 (0.77)		0.629	0.588	15.28

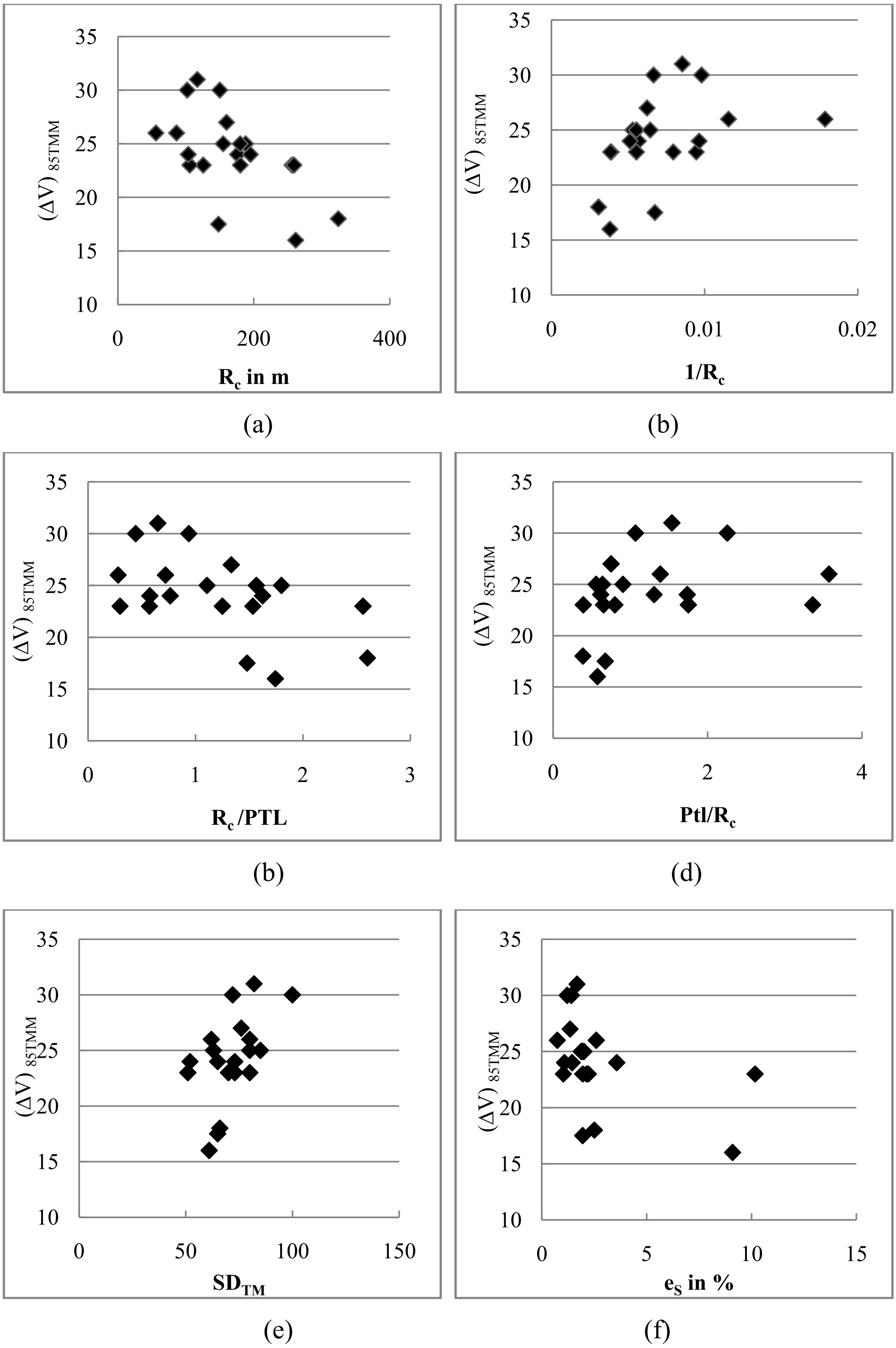


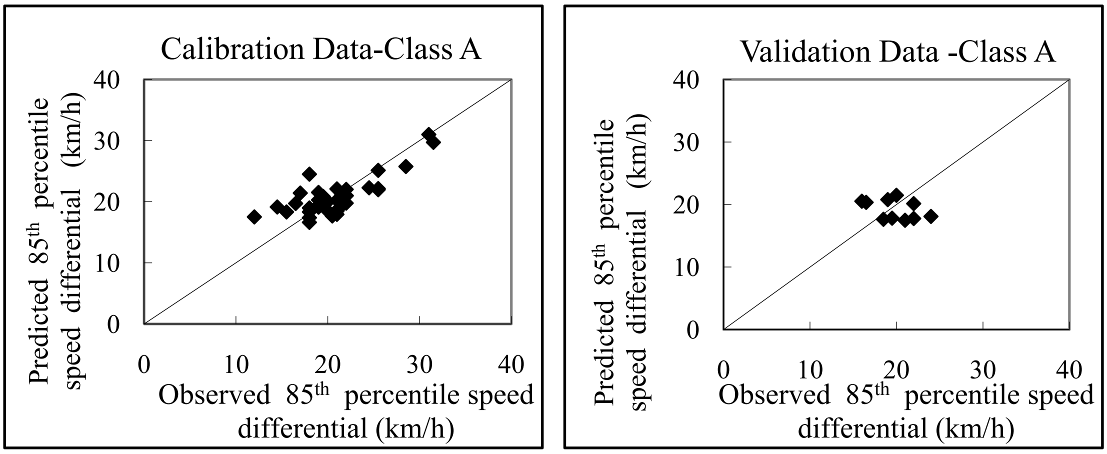
Fig.3.31 Relationship between 85th speed differential and significant variables for the Class C

Table 3.31 Speed differential models for the intermediate lane rural highways of Class C curves

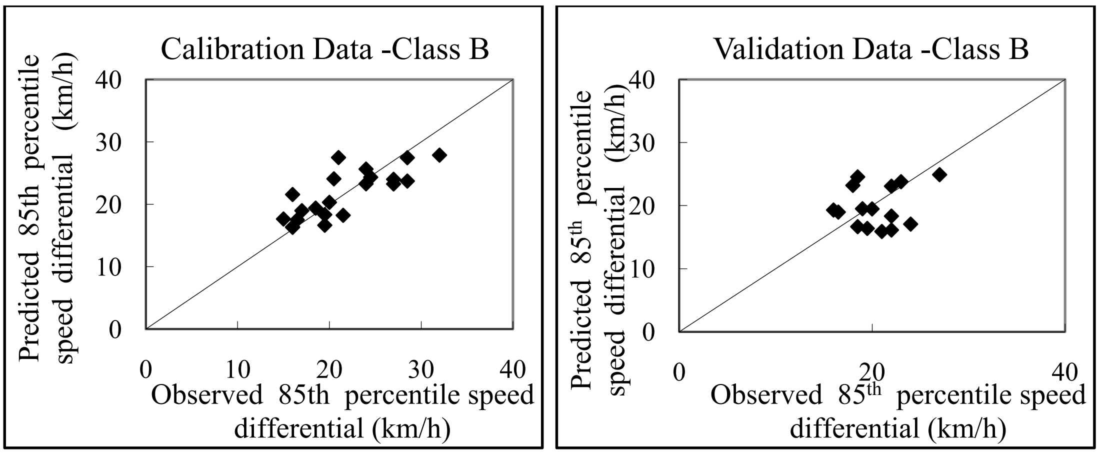
Model No.	Constant (t-value)	Coefficient of explanatory variables (t-values)							R^2	Ra^2	F-value
		R_c	$1/R_c$	Δ	PTL/R_c	R_c / PTL	SD_{TS}	$\sqrt{R_c}$			
1	29.684 (15.003)	-0.033 (-2.996)		-	-	-	-		0.333	0.296	8.97
2	20.78 (10.78)		475.277 (1.942)		-	-	-		0.173	0.127	3.77
3	34.551 (9.176)							-0.820 (-2.811)	0.305	0.266	7.90
4	19.302 (9.711)			0.134 (2.65)					0.281	0.241	7.032
5	22.428 (15.81)				1.403 (1.48)				0.109	0.060	2.203
6	27.728 (17.68)					-2.98 (-2.59)			0.272	0.232	6.72
7	11.648 (2.38)						0.176 (2.59)		0.272	0.232	6.739
8	16.95 (3.38)					-2.398 (-2.23)	0.141 (2.24)		0.438	0.372	6.63
9	18.345 (3.8)	-0.037 (-1.701)				0.895 (0.409)	0.153 (2.53)		0.524	0.435	5.87
10	22.537 (3.923)						0.150 (2.553)	-0.714 (-2.76)	0.498	0.439	8.4

3.11.4 Recommended speed differential models

Scatter plots, Figs.3.32 (a) and (b) were plotted to compare the observed and predicted speed differentials considering two classes of curves. For calibration and validation data, the Percentage Root Mean Square Error (PRMSE) values were calculated to check the goodness of fit of the models. From the PRMSE values, one can conclude that the speed prediction models are good enough in predicting the 85th percentile speed differentials. Also, both observed and predicted speed differentials are near to 45° line. Table 3.32 presents the recommended speed differential models for class A and class B curves.



(a) Class A curves



(b) Class B curves

Fig.3.32 Comparison between observed and predicted speed differential values

Table 3.32 Recommended Speed differential models

Class	Model	R^2	PRMSE	
			Calibration	Validation
A	$(\Delta V)_{85TMM} = 15.746 + \dfrac{4.797\ PTL}{R_c}$	0.589	12.1	16.9
B	$(\Delta V)_{85TMM} = 11.522 + \dfrac{0.711PTL}{\sqrt{R_c}}$	0.617	13.8	18.7

The aim of this study is to find out the influence of geometric variables in the prediction of operating speed and speed differentials. The developed operating speed and speed differentials models of intermediate lane rural highways indicate that in addition to geometric characteristics, other factors, too, may have an influence.

3.12 DEVELOPMENT OF CONSISTENCY EVALUATION CRITERIA FOR HORIZONTAL CURVES

The safety and comfort that the roadway provides to users are linked to a number of factors that involve the roadway features and driver behavior; therefore, the design consistency is important because of its relationship with safety (McCarthy 2011). Geometric design consistency evaluations are a widely used method of determining sections of highways which require alignment improvement. This method identifies geometric inconsistencies on highways by means of design evaluation criteria.

The change in speed of the vehicles is a visible indicator of inconsistency in geometric design (Nicholson 1998). Also, in the literature, the most commonly used criteria to evaluate the highway design consistency are based on operating speed. In addition to this, in this study, other approaches are also tried to develop best evaluation criteria. Inconsistent highway locations need not necessarily be high accident locations (Nagaraj et al. 1990), but may cause discomfort to drivers/passengers. Hence such alignment locations need to be eliminated while designing the alignment.

During the process of identification of the location of accident site it was observed that many accidents occurred at site were not recorded and that the identified accidents were scattered. With the help of limited accident details, several methods are tried to develop design consistency criteria for intermediate lane rural highways. The approaches were developed by comparing geometric variables, operating speed, and speed differentials with accident data. Consistency evaluation criteria can be developed for curve itself or successive elements of a curve along the stretch.

3.12.1 For Single Element

Following are the four methods tried, to develop consistency evaluation criteria for a single geometric element in addition to the method available in the literature (Lamm 1988, 1996).

3.12.1.1 Based on 85th percentile speed

It is hypothesised that higher the operating speeds than the average speed at the middle of the curve higher will be the chances of occurrence of accidents. Therefore, the observed operating speed at middle of curve was classified with different threshold values. The total number of accidents were classified as 0, 1 and >2 and type of accidents as fatal, grievous and simple injury. The classified threshold values of operating speed at the middle of the curve were compared with classified range of accidents and their severity. The number of curves matching with each category was counted. Among various trials with different ranges of operating speed, the best results (16.1 % i.e. 25 curves out of 155 horizontal curves) were obtained with the classified ranges of operating speed of <40 km/h, 40-50 km/h and >50 km/h. The developed consistency matrix for this case is shown in Fig.3.33.

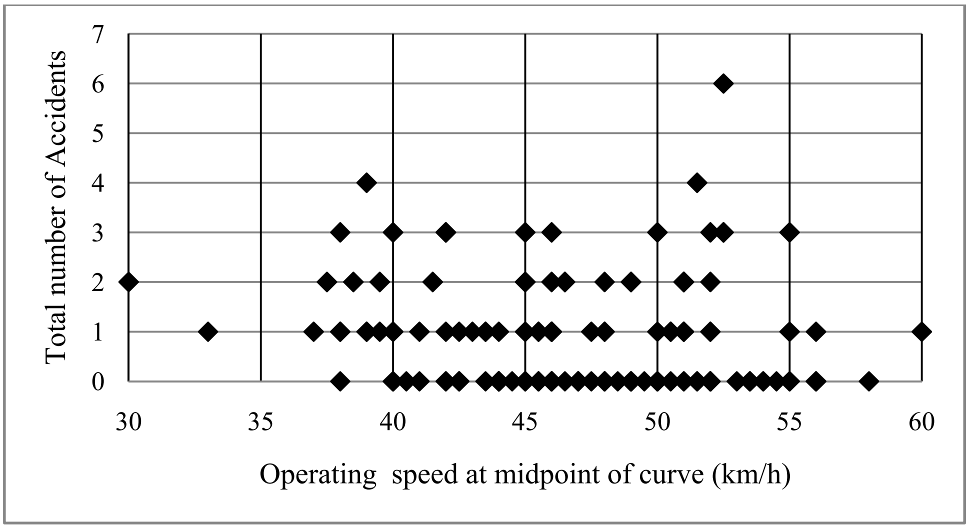


Fig. 3.33 Consistency matrix developed for operating speed, V_M

3.12.1.2 Based on deficiency of sight distance at start of curve

In this method, lack of sight distance available at the start of curve has been used to measure the deficiencies of different highway geometric designs. Sight distance required at the start of curve is calculated using the formula,

$$\text{Stopping sight distance, SSD}=0.278V_S\,t + \frac{V_S^{\,2}}{254\,f} \qquad \text{--- (3.5)}$$

Where,

V_S = Operating speed at start of curve

f = Coefficient of friction (0.35-0.4) (depends on operating speed at start of curve)

t = Reaction time of a driver=2.5 sec

The difference between available sight distance and required sight distance at the start of curve was calculated. Then the deficiencies of sight distance at the start of curve were classified into three categories. The three sets of classification are only to develop the consistency measure as poor, satisfactory, and good. The curves falling under each category (range of deficiencies of sight distance values) were compared with curves classified based on accident data. Among various trials performed with various ranges of deficiency in sight distance at start of curve, a maximum (23.2%) of curves matched the classification <20 m, 20-25 m and >25 m with accident data. The developed consistency matrix is as shown in Fig. 3.34.

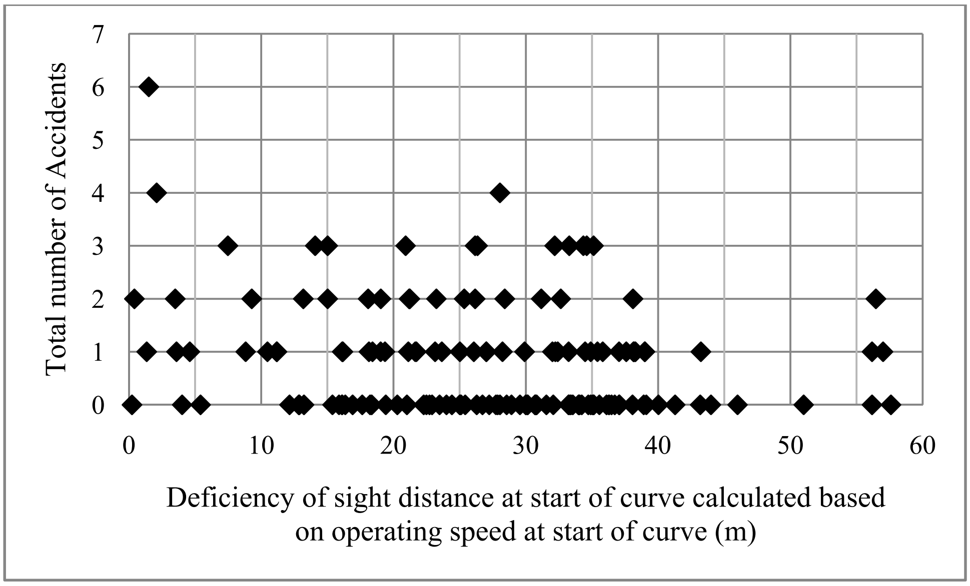


Fig. 3.34 Consistency matrix developed for deficiency of sight distance at start of curve

3.12.1.3 Based on deficiency of sight distance at study points

In this method, deficiency of sight distance at the start, mid, and end of curve has been used as a design consistency measure. The required sight distance at different study points i.e. at start, mid and end of curve were calculated for the design speed of highway (80 km/h) using the Eq.3.5. Then the difference between available sight distance and required sight distance was calculated at different study points separately. The curves were classified based on deficiency of sight distance at different study points of curve with three set of threshold values, and compared with accident data. The curves falling under each range were identified by developing consistency matrix as shown in Fig.3.35. The best results obtained from the trials based on deficiency in sight distance at different study points of curve, show less than 30%.

3.12.1.4 Based on design speed

Design consistency evaluation of independent highway elements was done by finding the relationship among number of accidents and type of accident with the difference between design speed and operating speed.

The design speed at the mid of the horizontal curve was calculated using

Design Speed, $V_d = \sqrt{127R_c(e_M + f)}$ ---- (3.5)

Where,

e_M= Observed superelevation at the centre of curve

R_c= Radius of curve in m

f = Coefficient of lateral friction = 0.15

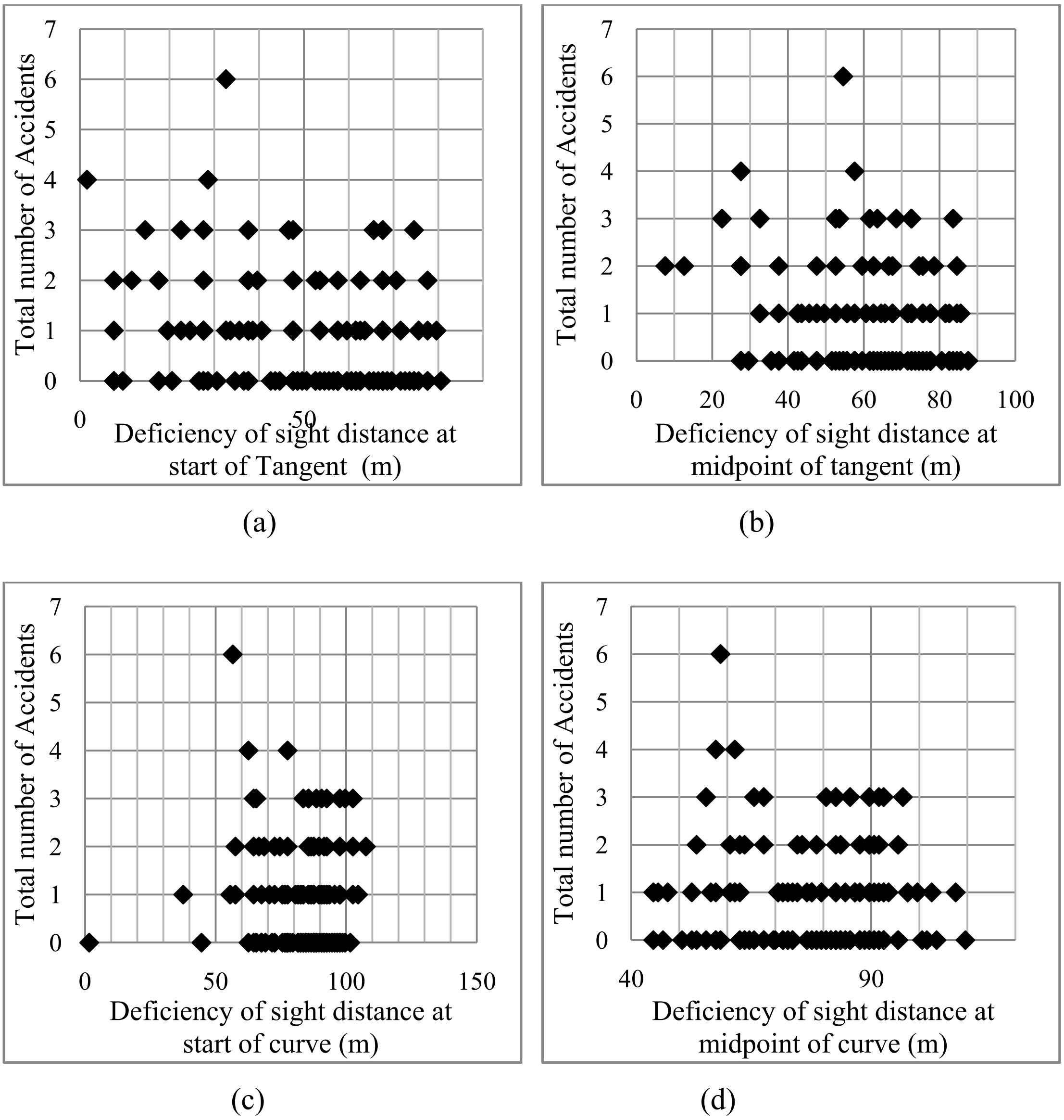


Fig. 3.35 Consistency matrix developed for deficiency of sight distance at study points calculated based on design speed of highway.

Then the difference between observed operating speeds at middle of curve and design speed (ie, V_M-V_d) was calculated. This value $|V_M-V_d|$ is compared with classified accident data by developing consistency matrix as shown in Fig.3.36. The curves under each class were identified and many such trials were performed to find maximum matching results. Among various trials of $|V_M-V_d|$ best results (33.5% i.e. 52 curves out of 155 horizontal curves) were obtained, with ranges of $|V_M-V_d|$ being <10 km/h, 10-18 km/h and >18 km/h.

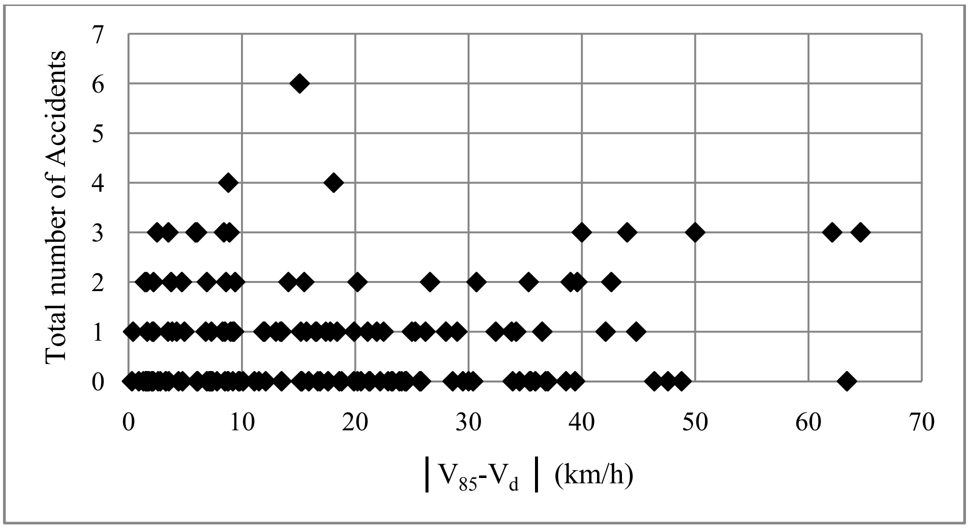


Fig.3.36 Consistency matrix developed for $|V_{85}-V_d|$

Among four speed based consistency evaluation methods developed better results are obtained with $|V_M-V_d|$. Therefore, the consistency evaluation criteria based on $|V_M-V_d|$ is recommended for consistency evaluation of single element of intermediate lane rural highways. The developed consistency evaluation criteria are tabulated in Table 3.33.

Table 3.33 Recommended consistency evaluation criteria - Single element

Criterion	Consistency measure	Evaluation
$\|V_M-Vd\|$	0-10 km/h	Good
$\|V_M-Vd\|$	10-18 km/h	Satisfactory
$\|V_M-Vd\|$	> 18 km/h	Poor

In this study it is concluded that a good design consistency can be achieved when the difference between operating speed and design speed of curve is less 10 km/h. If this difference is more than 18 km/h then there exists an inconsistency, which leads to unusual situations. This criterion is almost similar to the criteria developed by Lamm et al. (1988, 1996) for single element of two lane rural highways. From the results, it is clear that the manoeuvres are caused by the driver's need to suddenly correct his or her driving behaviour due to an unexpected alignment and can produce a dangerous situation if unexpected events occur.

3.12.2 For Successive Elements

3.12.2.1 Based on 85th speed differential

A geometric design consistency evaluation criterion is developed based on speed differential for intermediate lane rural highways. The 85th speed differential reflects the 85th percentile maximum speed reduction between two successive highway elements (tangent to middle of curve), as experienced by the same vehicle or driver. The 85th speed differential value of curves was compared with classified accident data by developing the consistency matrix as shown in Fig.3.37.

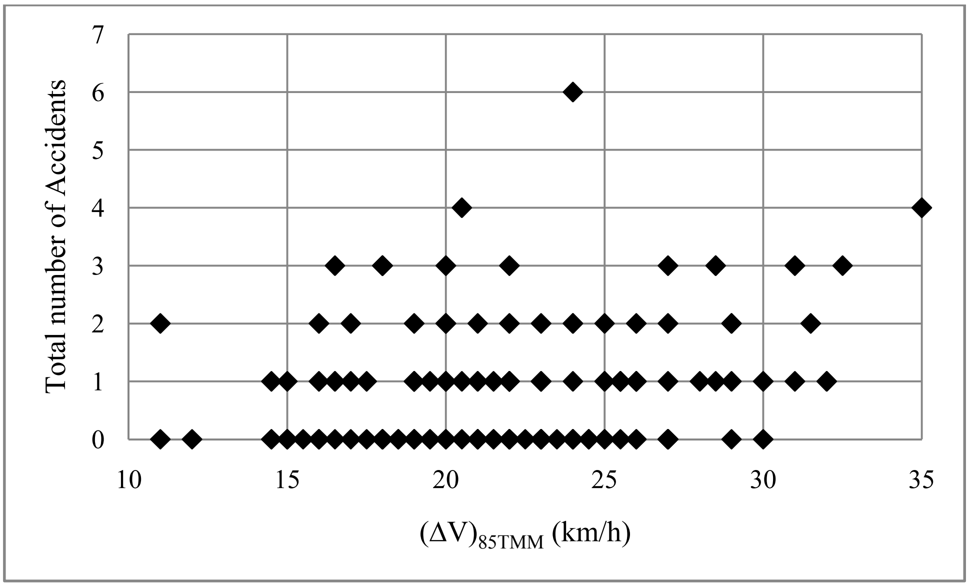


Fig.3.37 Consistency matrix developed for 85th speed differential

Numbers of trials were performed with different ranges of speed differentials and history of accidents to find the best results. The following Table 3.34 shows the consistency evaluation criteria developed based on the best results obtained by comparing 85th percentile speed differential and accident data. The result shows that 33.5% (52 curves out of 155 horizontal curves) curves agree with the developed consistency evaluation criteria.

From the Table 3.34, it can be observed that the threshold values of the criteria developed based on speed differential and accident data of intermediate lane rural highways show higher than the threshold values of Lamm's criteria developed for successive elements of two lane rural highways.

Table 3.34 Consistency evaluation criteria based on speed differential

Criterion	Criterion	Evaluation
$(\Delta V)_{85TMM}$	0-20 km/h	Good
$(\Delta V)_{85TMM}$	20-25 km/h	Satisfactory
$(\Delta V)_{85TMM}$	> 25 km/h	Poor

3.12.2.2 Based on operating speed

The simplest and most common method to evaluate design consistency of successive elements is based on operating speed values (Lamm et al. 1988). In this case, the difference in operating speeds between two successive elements, tangent section and mid of the curve ($\Delta V_{85}= V_{TM}-V_M$) was calculated. A number of trials were performed with different ranges of difference in operating speed and compared with accident data and consistency matrix was developed as shown in Fig.3.38. Among different sets of speed ranges tried, 0-15 km/h, 15-20 km/h, and > 20 km/h gave good results with number of accidents. Out of the 155 horizontal curves, 77 horizontal curves (49.6%) were correctly classified based on the developed consistency evaluation criteria.

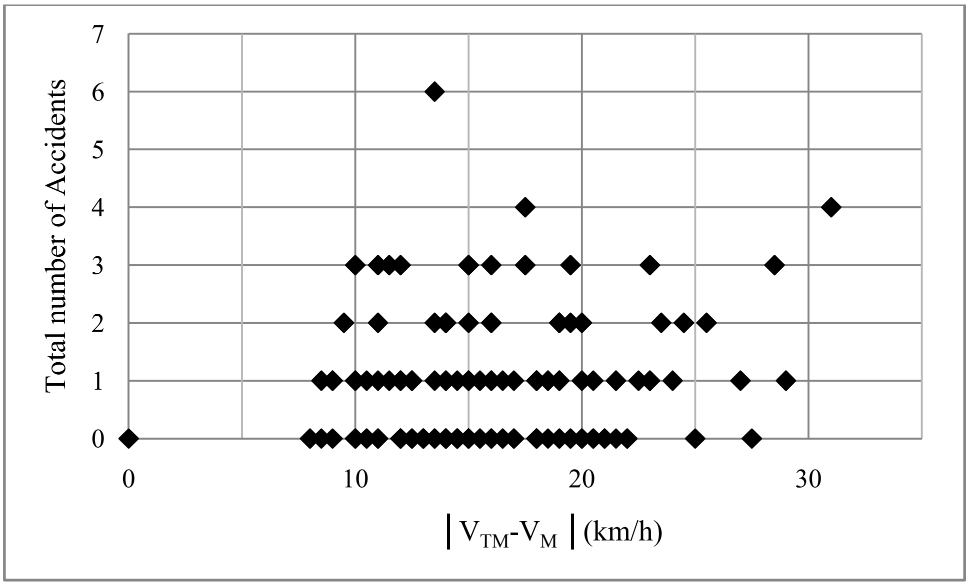


Fig.3.38 Consistency matrix developed for │V_{TM}-V_M│

The criteria developed based on operating speed give better results than speed differential. Hence the consistency evaluation criteria based on │ V_{TM}-V_M │are recommended for consistency evaluation of successive elements of intermediate lane rural highways. The developed consistency evaluation criteria are tabulated in Table 3.35.

Table 3.35 Recommended consistency evaluation criteria-Successive elements

Criterion	Criterion	Evaluation
$\Delta V_{85} = V_{TM}-V_M$	<15 km/h	Good
$\Delta V_{85} = V_{TM}-V_M$	15-20 km/h	Satisfactory
$\Delta V_{85} = V_{TM}-V_M$	> 20 km/h	Poor

From this study it is concluded that a good design consistency can be achieved when the difference between operating speed on the tangent and the following curve does not exceed 15 km/h. Which is slightly greater than the recommended value of 10 km/h by a Kannallaidis et al. (1990) and Lamm et al. (1988,196). The criteria recommended for poor design is very similar to Lamm et al. (1988,1996). Therefore it can be concluded that as the differences between the two factors that characterize the various safety criteria increase, there is a progressive decrease in the degree of consistency and thus a probable increase in dangerous situations.

The consistency evaluation criteria recommended for single element and successive elements of Class A, Class B and Class C curves of intermediate lane rural highways are same as tabulated in Table 3.33 and Table 3.35 respectively.

In this study, 49.6% of accidents occurred on curves are matched with developed consistency criteria based on conventional ΔV_{85} methodology, whereas only 33.5% of accidents occurred on curves are matched with the speed differential $((\Delta V)_{85TMM})$ method. This indicates that, conventional ΔV_{85} methodology is more appropriate than speed differential method for the design consistency evaluation of intermediate lane rural highways.

3.13 SUMMARY

In this chapter, the data collection methodology for consistency evaluation of horizontal curves is discussed. Data exploration was done based on accident data collected on intermediate lane rural highways and identified accidents on horizontal curve, to identify the reason for accidents. From the analysis, it was found that curved sections are the most critical locations required to be considered for the improvement of highway safety. The horizontal curve with a radius of less than 250 m is very dangerous because number of accidents and severity of accidents are more. It is found that radius and available sight distances at curve are the significant variables affecting the prediction of operating speed. Preceding tangent length and radius are the significant variables affecting the prediction of 85^{th} percentile speed differential. Carriageway width and shoulder width are the parameters which controled the design consistency evaluation of intermediate lane highways. The results show that lack of speed consistency has been used to measure the deficiencies of different highway geometric designs in intermediate lane rural highways. Potential design inconsistencies result from exceeding the design speed on a specified curved section or from significant difference in operating speeds on two successive sections. The consistency evaluation criteria developed for single element and successive elements of horizontal curve is almost very close to Lamm's criteria.

CHAPTER 4

CONSISTENCY EVALUATION OF VERTICAL CURVES

4.1 INTRODUCTION

The study on the effect of vertical alignment on safe operation of traffic is necessary. In order to develop a speed prediction model it is necessary to gather data on vehicle speeds. This was done by selecting a number of sites on intermediate lane rural highways and measuring the speeds of a sample of vehicles. This chapter discusses the data collection methodology in detail. The operating speed models including their validation for vertical summit curves are also discussed. An attempt is made to develop consistency evaluation criteria for vertical summit curves.

4.2 DATA REQUIREMENT

On review of literature, it is found that the basic data requirements for the consistency evaluations of vertical curve are 1) Geometrics 2) Accident and 3) Speed data. In this study the geometric data include the details such as length of vertical curve, approach grade, departing grade, algebraic difference of grades, rate of vertical curvature, preceding tangent length etc. The speed data include speed of passenger cars at different locations of each site. Accident data include the type and location of accident.

4.3 SITE SELECTION

Several field visits were made to identify the stretches with vertical curves. As sufficient number of crest vertical curves are available on the selected stretches of intermediate lane rural highways, this study is limited to crest vertical curves. Variation in roadside conditions may have adverse affects on the operating speed of vehicles; hence, the sites for vertical curves are selected by considering site selection criteria. The same site selection criteria considered for horizontal curves (as in section 3.3.2) was also selected for vertical curves. Additional criteria used in selecting the sites include the rate of vertical curvature (K) . K (m/%) was limited to less than or equal to 43 m/%, so that the vertical curves have a limited sight distance (Fitzpatrick et al. 2000a).

The topography of the project area ranges from rolling to mountainous terrain. Based on site selection criteria, a total of 40 crest vertical curves with more than 100 m preceding tangent length were considered. Numbers of crest vertical curves available in the selected eight rural highways are tabulated in the Table 4.1.

Table 4.1 Study curves considered in the selected stretches

Road	Number of Curves selected for study	From	To
SH -37	1	Subrahmanya	Belthangadi
SH -64	4	Charmadi	B.C. Road
SH -67	-	Permude	Maradka
SH -70	3	Belthangadi	Mulki
SH- 88	21	Sampaje	Puttur
SH -101	2	Polali	Bajpe
SH-114	4	Kulkunda	Gundya
NH- 13	5	Nantoor	Yadapadavu
Total	40		

4.4 DATA COLLECTION

4.4.1 Geometric data

Theodolite surveying was conducted to obtain the geometric details of the study sites. The collected details were used to prepare CAD drawings. Using these CAD drawings, the required geometric data of all the curves were retrieved. The geometric data required for this study include information about the crest vertical curve and the tangent preceding these curves, i.e. the length of vertical curve, approach grade, departing grade, algebraic difference of grade, rate of vertical curvature, preceding tangent length, departing tangent length, length of approach grade etc. The details collected at study points on the vertical summit curve are shown in Fig 4.1.

For the vertical summit curves with limited sight distances (LSD), points AT, LP and SP are speed observation points, as shown in the Fig.4.1. TM is the approach tangent point located on the tangent section. LP is the limit point where the stopping sight distance is minimum, while SP is the crest or summit point of the vertical curve.

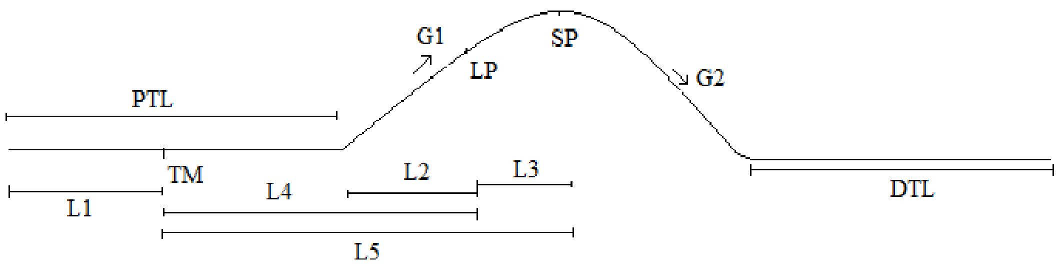


Fig.4.1 Features of a crest vertical curve

In this study, limit point was obtained considering height of the driver's eyes, h_1=1.2 m and the height of the obstruction, h_2=0.15 m as per IRC guidelines. A number of trials were performed to obtain limit point of crest verical curve in the CAD drawings of sectional plan of site. The line of sight joining the top of h_1=1.2 m, positioned at varying distance from the summit on the ascending grade section, and the summit point was extended to intersect h_2=0.15 m positioned on the descending grade section. The corresponding distance between h_1 and h_2 was noted. The location of $h_{1,}$ where the sight distance is found to be the least, was considered as point LP. Jessen et al. (2001) observed that the location of LP ranged from 46 m to 122 m from crest on the vertical curves of two lane rural highways. In this study the distance of LP from the summit (SP) was observed to be varying between 20-55 m. The detail of sight distance available at the start of tangent (SD_{TS}) was also collected. The other variables considered for crest vertical curves are described below:

4.4.1.1 Length of the vertical curve (L_V)

The length of the vertical curve, L_V (m) is the distance between the start and end points of the vertical curve. In this study, the horizontal projection between the start and end points of the vertical curve is considered equal to the actual length of the vertical curve. The length of the vertical curve is obtained from CAD drawing.

4.4.1.2 Approach grade (G1) and Departing grade (G2)

Grades have a physical effect on speeds (Fitzpatrick et al. 2000a, Jessen et al. 2001). Gradient is the rate of rise or fall along the length of the road with respect to the horizontal. It is expressed either as a ratio or 1 in x (1 vertical to x horizontal) or as a percentage, n (i.e. n in 100). The ascending gradients are denoted with positive signs and descending gradients are denoted with negative signs. For this study, gradient is expressed in percentage and was obtained from the vertical alignment of a road as a ratio of difference of levels to the horizontal length of a section.

4.4.1.3 Algebraic difference of grades, A

'A' is calculated as the algebraic difference between approach grade (G1) and departing grade (G2).

$$A = G1-(\pm G2) \quad \text{-- (4.1)}$$

4.4.1.4 Rate of vertical curvature (K)

Crest vertical curves on horizontal tangents were divided into those with limited sight distance (K<43 m/%) and non-limited sight distance (K>43 m/%). The operating speeds on crest vertical curves with LSD to be a function of the rate of vertical curvature (Fitzpatrick et al. 2000a). The rate of vertical curvature, K was determined using the following equation:

$$K = \frac{L_V}{|A|} \quad \text{-- (4.2)}$$

Where,

K = rate of vertical curvature (m/%)

L_V = Length of vertical curve (m)

This study is restricted to crest vertical curves with limited sight distances.

4.4.1.5 Preceding tangent length (PTL) and Departing tangent length (DTL)

A long straight section creates inert state of mind, and also creates psychological feeling in the driver to speed up, thus creating a hazardous situation. Considering this criteria straight section before and after the vertical curve is considered for the study. The Preceding Tangent Length (PTL) is the straight levelled portion of road before the start of vertical curve and Departing Tangent Length (DTL) is the straight levelled portion of road after the end of vertical curve. The approach tangent length and departing tangent length are measured from the CAD drawing of section plan of vertical curve.

4.4.1.6 Length of approach grade (LAG)

Length of a grade or the approach to a crest vertical curve is of importance because its combination with steep grades can affect speeds (Fitzpatrick et al. 2000a). Therefore in addition, depending upon the position of the points on the vertical curve where speed observations were taken, the following distances between speed observations point were extracted from the CAD drawing.

(i) L1=Tangent length between the end of the preceding curve and the approach tangent point, AT in m.

(ii) L2=Length of the ascending grade section up to limit point, LP or the distance between the start point of vertical curve and LP, in m.

(iii) L3=Length between points SP and LP, in m.

(iv) L4= Length between points AT and LP, in m.

(v) L5=Length between points AT and SP, in m.

(vi) LLS= Length of grade between LP and SP, in m.

$$= \sqrt{L3^2 + \left(\frac{G1x\,L3}{100}\right)^2} \qquad \text{-- (4.3)}$$

(vii) LAG= Length of ascending grade up to SP, in m.

$$= \sqrt{(L2 + L3)^2 + \left(\frac{G1x\,(L2 + L3)}{100}\right)^2} \qquad \text{-- (4.4)}$$

The geometric details of all selected vertical curves are tabulated in Table 4.2.

4.4.2 Speed Data

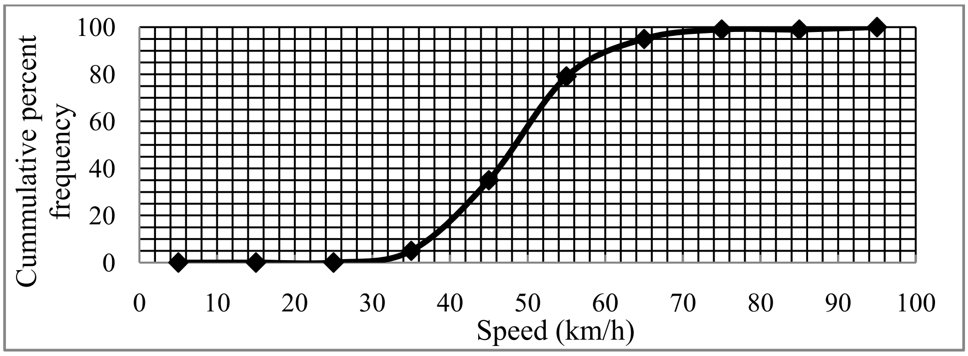
The main traffic characteristics of interest for design consistency evaluation of vertical curve is the speed of vehicle. The other factors of interest were the time of day and pavement condition (wet/dry) (Jessen et al. 2001). Lamm et al. (1990) found insignificant difference between day versus night speeds and wet versus dry pavement conditions, respectively. From the study Fitzpatrick et al. (2000a) concluded that effects of weather (dry versus wet) or light (day versus night) have a significant large effect on two lane rural highways and thereby impact on design consistency evaluation. In this study, the speeds were measured during daylight, off-peak periods, and under dry weather conditions. For all of the field studies, at least 100 passenger car's speeds were collected manually at all selected points to ensure an adequate sample size to obtain a 95% level of confidence under free flow conditions. At each point, a trap length of 10 m was considered and the time taken for each vehicle to traverse the trap was noted. Observers were located at each point in such a way that their presence would not influence the speeds of passing vehicles. The speeds were measured at three points on the vertical summit curve with limited sight distances as follows:

- Point 1 (TM): At middle of the tangent section.
- Point 2 (LP): Before the crest, where sight distance is limited.
- Point 3 (SP): On the crest or summit point.

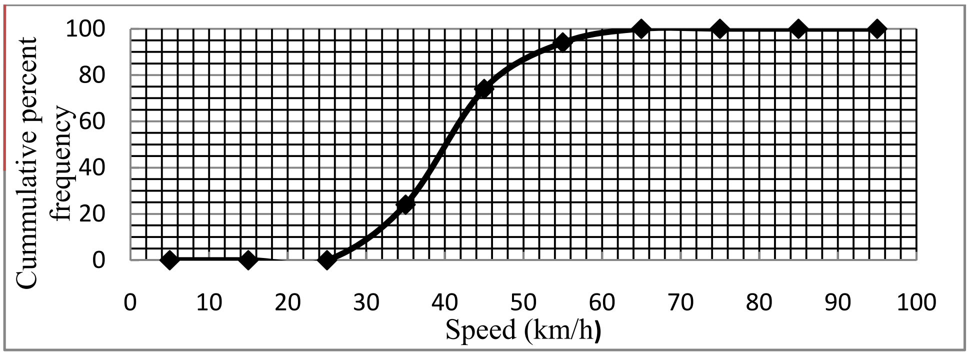
These points are shown in the Fig.4.1.

4.4.2.1 Estimation of operating speed

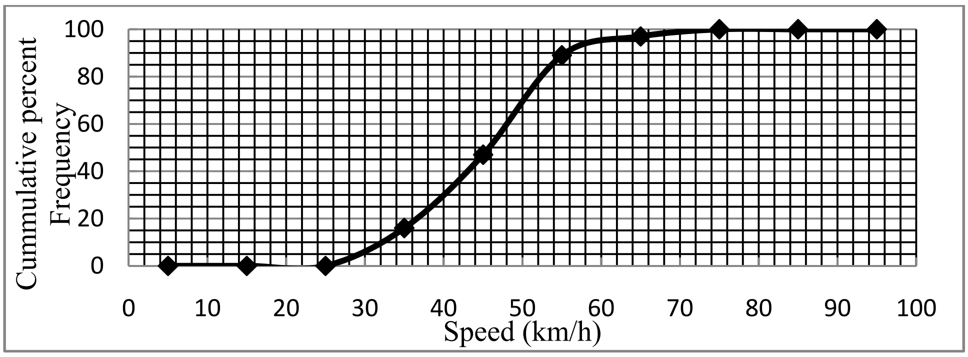
Operating speed is defined as the speed selected by the highway users when not restricted by other users, i.e., free-flow conditions, and is normally represented by the 85th percentile speed (V_{85}) (Gibreel et al. 1999, Fitzpatrick et al. 2000a). Time taken by each vehicle to traverse the trap was entered in a spreadsheet. Then spot speed of each vehicle was calculated. Similarly, for each set of speed observations at three different points on each selected curve, a calculation spreadsheet was made use of. Then the observed speeds were evaluated for sufficiency of the sample size at 5 % significance level and allowing a permissible error in speed of ± 2 km/h. To obtain the operating speed, V_{85} cumulative percent frequency distribution table was prepared, based on the observed speeds. The spot speed values were grouped using the class interval estimated. Class interval was found to be 5 km/h in most of the cases. The cumulative percent frequency was estimated for each class. Cumulative percentage frequency plot was prepared. The speed value corresponding to 85th cumulative percent was found from this plot. A typical cumulative percent frequency graph at different study points of vertical curve, V21 are shown in Figs. 4.2(a) to (c).



(a) Operating speed at tangent point of V21



(b) Operating speed at limit point of V21



(c) Operating speed at summit point of V21

Fig 4.2 Cumulative percentage frequency distribution of speeds

The values of observed operating speeds (85th percentile speed) of vertical curves at three different points: i.e. on tangent section (V_{TM}), at limit point of curve (V_{LP}) and summit of the curve (V_{SP}) are tabulated in Table 4.2. From Fig. 4.3, it can be noted that the drivers tend to decrease their speed while approaching the limit point of the curve, where the sight distance is most limited and then increase as vehicle approaches the summit point of curve.

4.4.2.2 Trend of operating speed at study points of crest vertical curve

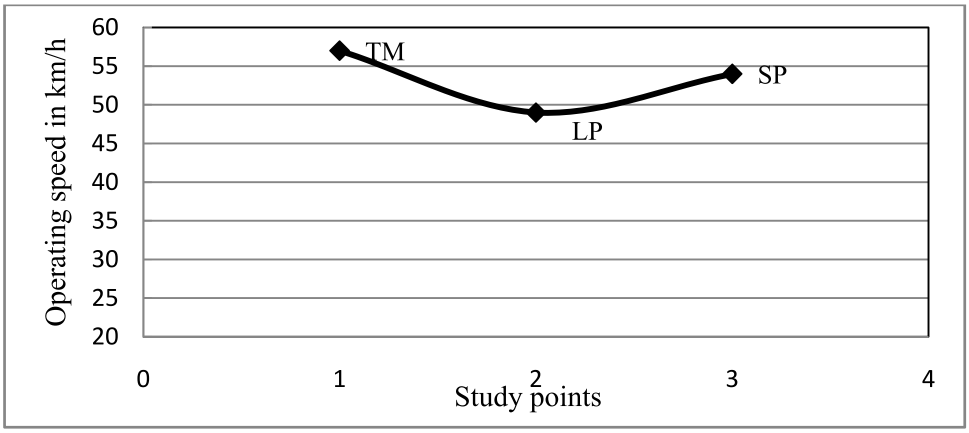


Fig. 4.3 Observed trend of operating speed at study points of V21

A plausible explanation of this trend is that while moving from tangent section to the limit point of curve, the sight distance decreases along with gradient of road and this sight distance restriction with gradient forces the drivers to reduce their speed. However, on approaching the summit point of vertical curve, the sight distance starts to increase and drivers tend to increase their speed.

4.4.3 Accident Data

A detailed methodology adopted for accident data collection and location identification was explained in section 3.6. A pilot survey was conducted to identify accident spots along the stretches with the help of a police person and two to three local persons to identify FIR (First Information Report) registered accident spots. The type of injury involved was noted on the alignment map for the corresponding location as tabulated in the Table 4.2. Even though six years (2005-2010) of accident data were collected, a total of 23 reported accidents of last three years were identified on crest vertical curves.

4.5 DEVELOPMENT OF SPEED MODELS FOR CREST VERTICAL CURVES

The main measures of operating speed design consistency for two lane highways have been summarised in Chapter 2. Being equipped with those measures, how to predict the operating speed accurately for intermediate lane rural highway becomes a key challenge. In this study the regression analysis technique was employed for model estimation. Many trials were performed and only the most significant and logical models are presented.

4.5.1 Development of Operating Speed Models

Of the 40 curves for which details were collected, 26 data sets were used to develop the relationships, and the additional 14 data sets were used for verification. Influential curves were identified and removed to improve the reliability of the models. A total of two curves out of 26 curves were removed, leaving 24 curves for the analysis procedures. These 24 curves were used for model development. Scatter plots and correlation matrix were then used to identify the most significant independent variables. Pearson correlation coefficients at different study points of vertical curve are given in Table 4.3. High correlation coefficient shows a better relation with pair of variables. The positive sign of the correlation coefficient indicates a direct relation and a negative sign indicates an inverse relation with operating speed.

At selected points, the best operating speed model selection process was used to identify influential variables by performing multiple linear regression method. The developed operating speed models and those satisfying the F-test, t-tests and model logic were noted down. Finally the best models under each category were selected based on the coefficient of regression R^2, F-value and t-values.

Table 4.2 Data of Vertical Curves

Stretch No.	Curve	L_V (m)	G1 (%)	G2 (%)	A (m)	K m/%	PTL (m)	DTL (m)	L1 (m)	L2 (m)	L3 (m)	L4 (m)	L5 (m)	SD_{TS} (m)	V_{TM} km/h	V_{LP} km/h	V_{SP} km/h	LAG (m)	LLS (m)	Type of Accidents		
																				FA	GI	SI
2	V1	395.0	5.0	-4.4	9.4	42.0	300	100	150	171	55	321	376	95	62	35	34	226	55.1			
6	V2	100.0	2.5	-3.2	5.7	17.5	120	60	60	20	50	80	130	65	55.5	50	52	70	50			
7	V3	120.0	3.7	-3.2	6.9	17.4	100	90	50	50	35	102	137	55	51	41	45	85.1	35		1	
9	V4	150.0	3.8	-3.0	6.8	22.1	110	34	50	40	34	100	134	60	53	43	45	74.1	34			
10	V5	200.0	3.5	-2.5	6.0	33.3	120	90	55	87	35	152	187	60	54	44	46	122	35	1		
10	V6	170.0	3.6	-2.5	6.1	27.9	110	40	50	73	35	133	168	60	49	38	40	108	35			
11	V7	110.0	4.8	-4.0	8.8	12.5	100	50	50	47	29	97	126	55	52	42	46	76.1	29			
11	V8	200.0	3.3	-2.4	5.7	35.1	140	70	65	67	38	142	180	60	58.5	50	51	105	38			
11	V9	190.0	4.3	-1.7	6.0	31.7	160	60	70	110	30	200	230	65	61	42	44	140	30		1	
12	V10	115.0	2.7	-2.4	5.1	22.5	150	105	70	17	46	97	143	60	54	52	53	63	46			
12	V11	90.0	3.0	-1.5	4.5	20.0	150	60	70	33	43	113	156	60	55	48	50	76	43		2	
12	V12	180.0	3.0	-2.5	5.5	32.6	255	185	150	61	35	166	201	85	61	44	46	96	35	1		1
14	V13	240.0	3.5	-2.2	5.7	42.1	110	83	50	142	35	202	237	55	50	40	44	177	35			1
14	V14	237.0	4.9	-2.8	7.7	30.8	165	83	80	104	22	189	211	55	55	44	46	126	22		1	
15	V15	378.0	4.1	-5.4	9.5	40.0	240	270	150	170	30	260	290	85	61	52	54	200	30		1	1
15	V16	270.0	3.5	-4.6	8.1	33.3	210	120	100	97	36	207	243	85	60	51	53	133	36			
15	V17	435.0	2.8	-5.2	8.0	54.4	210	250	100	120	40	230	270	85	60	50	53	160	40			
15	V18	260.0	3.4	-2.8	6.2	41.9	120	90	55	141	37	206	243	65	54	45	46	178	37		1	
16	V19	115.0	4.6	-1.3	5.9	19.5	100	56	50	41	29	91	120	54	54	40	42	70.1	29			
16	V20	102.0	3.8	-3.2	7.0	14.6	100	55	50	35	35	85	120	56	48.5	42	43	70.1	35			

Table 4.2 Data of Vertical Curves

Stretch No	Curve	L_V (m)	G1 (%)	G2 (%)	A (m)	K m/%	PTL (m)	DTL (m)	L1 (m)	L2 (m)	L3 (m)	L4 (m)	L5 (m)	SD_{TS} (m)	V_{TM} km/h	V_{LP} km/h	V_{SP} km/h	LAG (m)	LLS (m)	FA	GI	SI
17	V21	182.0	3.5	-2.1	5.6	32.5	140	40	60	72	36	152	188	60	57	50	55	108	36			
19	V22	100.0	3.7	-2.8	6.5	15.4	100	59	50	38	35	88	123	55	51	43	47	73	35			
19	V23	150.0	4.2	-3.4	7.6	19.7	170	58	75	52	30	147	177	75	57.5	40	44	82.1	30		2	
19	V24	220.0	5.5	-3.0	8.5	25.9	140	80	60	95	30	175	205	65	59	35	34	125	30		2	
20	V25	215.0	3.7	-3.3	7.0	30.7	120	27	50	71	35	141	176	60	52.5	48	47	106	35			
20	V26	165.0	4.5	-2.0	6.5	25.4	175	100	80	84	25	179	204	70	59.5	50	52	109	25			
22	V27	140.0	5.5	-1.5	7.0	20.0	120	50	78	70	30	112	142	60	53.5	41	40	100	30			
22	V28	240.0	3.7	-3.7	7.3	32.9	100	60	50	90	35	140	175	55	49	48	55	125	35			
24	V29	200.0	2.9	-3.2	6.1	32.8	130	70	65	45	50	110	160	65	51.5	51	53	95	50			
24	V30	130.0	3.8	-1.5	5.3	24.5	145	70	60	10	40	95	135	65	56.5	45	47	50	40			
31	V31	160.0	2.8	-3.0	5.8	27.7	100	20	45	85	35	140	175	55	52	50	53	120	35			
33	V32	240.0	3.4	-2.4	5.8	41.4	110	25	50	121	39	181	220	55	51	49	51	160	39			
34	V33	120.0	3.5	-3.0	6.5	18.5	120	30	55	44	36	109	145	60	56	52	53	80	36			
35	V34	240.0	5.1	-2.5	7.6	31.6	110	50	50	95	25	155	180	55	53	39	40	120	25		1	
37	V35	335.0	4.4	-3.7	8.1	41.4	215	30	100	100	32	215	247	85	60	50	53	132	32			1
42	V36	185.0	3.9	-1.2	5.1	36.3	225	100	100	75	35	200	235	85	65	54	57	110	35		1	
43	V37	285.0	6.4	-1.4	7.8	36.4	120	175	55	189	31	254	285	65	55	30	29	220	31.1		2	1
48	V38	210.0	4.7	-2.8	7.5	28.0	125	45	65	60	40	120	160	60	54.5	44	47	100	40			
50	V39	235.0	3.3	-4.7	8.0	29.4	115	50	55	96	39	156	195	55	52	45	47	135	39		1	
55	V40	140.0	2.8	-2.1	4.9	28.6	100	10	50	54	36	104	140	55	60	56	57	90	36			

4.5.2 Operating Speed Models at Tangent Point

The correlation matrix developed in Table 4.3 and scatter plots in Fig 4.4 show that PTL, L1, SDT, √PTL, $PTL^{0.3}$, $PTL^{1.5}$, ln(PTL) and 1/PTL are the principal variables having a good correlation with operating speed at tangent point. It can be observed that operating speed at tangent point shows fruitful relation with nonlinear variable than linear variables.

Figs. 4.4 (a) and (b) show that operating speed generally increases with increase in PTL and L1.The rate of increase in speed is more for an increase in PTL up to 200 m and this rate gradually decreases beyond 200 m. Fig 4.4 (c) shows that the sight distance available at the start of curve also has an effect on V_{TM}. Figs. 4.4 (d) to (g) show that operating speed at tangent point shows a better nonlinear relation with preceding tangent length than linear relation with PTL. As PTL or L1 increases the visibility to the driver also increases; hence driver accelerates the vehicle lead to increase in operating speed. It is observed from Fig. 4.4 (h) that inverse of PTL is highly correlated with operating speed at tangent point. Observing the relation found in scatter plots and in correlation matrix some of the following models are developed.

V_{TM}= 44.079+0.078 PTL R^2=0.660 -- (4.5)

V_{TM}= 47.420+0.113 L1 R^2=0.497 -- (4.6)

V_{TM}= 37.155+0.281SD_{TS} R^2=0.537 -- (4.7)

$V_{TM}= 68.451-\frac{1753.909}{PTL}$ R^2=0.722 -- (4.8)

V_{TM}= 31.872+1.977$PTL^{0.5}$ R^2=0.684 -- (4.9)

V_{TM}= 15.593+9.027 $PTL^{0.3}$ R^2=0.692 -- (4.10)

V_{TM}= 48.137-0.004 $PTL^{1.5}$ R^2=0.631 -- (4.11)

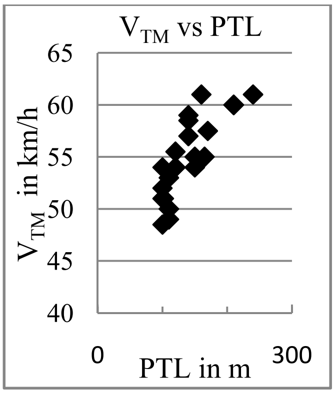
$V_{TM}= 96.963 -\frac{182.085}{PTL^{0.3}}$ R^2=0.711 -- (4.12)

V_{TM}= -4.848+12.208 ln(PTL) R^2=0.703 --(4.13)

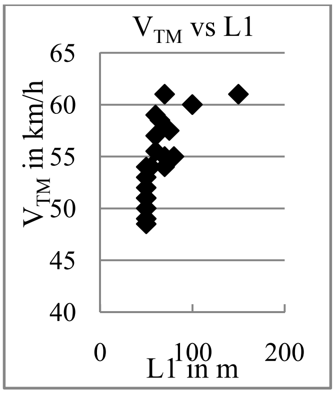
V_{TM}= 42.974+0.34 SD_{TS}+0.071PTL R^2=0.662 -- (4.14)

$$V_{TM} = 31.858 + 0.027\,SD_{TS} + 1.83PTL^{0.5} \qquad R^2 = 0.685 \qquad \text{-- (4.15)}$$

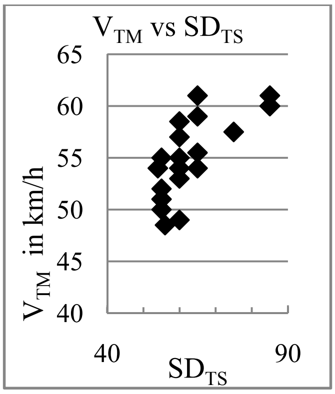
$$V_{TM} = 63.555 + 0.051SD_{TS} - \frac{1534.058}{PTL} \qquad R^2 = 0.728 \qquad \text{-- (4.16)}$$



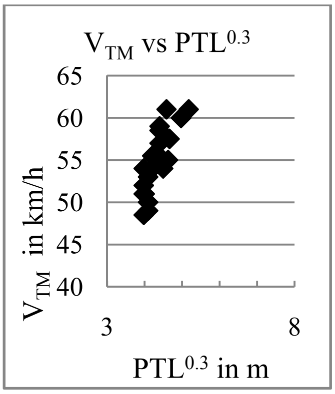
(a)



(b)



(c)



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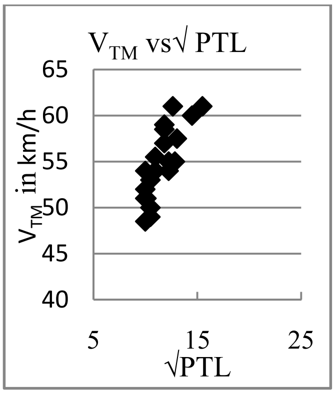

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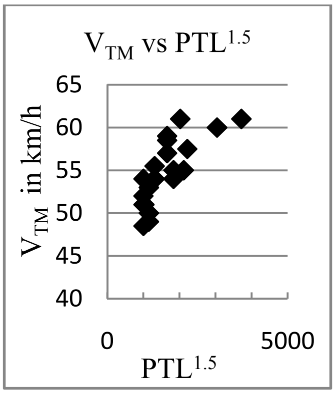

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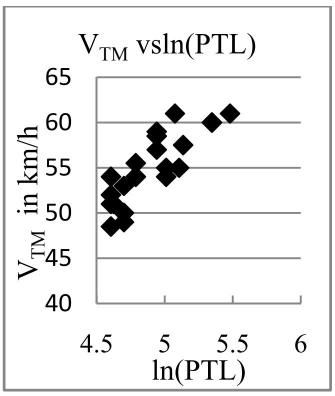
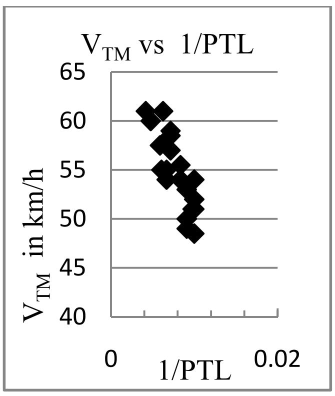


(g)



(h)

Fig.4.4 Relationship between operating speed at tangent and geometric variables

Table 4.3 Correlation matrix for Vertical curves

	LV	G1	G2	A	K	PTL	DTL	L1	L2	L3	L4	L5	SD_{TS}	V_{TM}	V_{LP}	V_{SP}	LGL	LAG	√PTL	$PTL^{0.3}$	$PTL^{1.5}$	Ln(PTL)	1/G1	√PTL / G1	1/ PTL
LV	1																								
G1	-0.01	1																							
G2	-0.61	0.03	1																						
A	0.50	0.56	-0.82	1																					
K	0.90	-0.22	-0.29	0.12	1																				
PTL	0.74	-0.07	-0.59	0.45	0.57	1																			
DTL	0.67	-0.33	-0.51	0.24	0.62	0.48	1																		
L1	0.71	-0.04	-0.66	0.53	0.49	0.95	0.37	1																	
L2	0.86	0.19	-0.36	0.41	0.83	0.53	0.31	0.54	1																
L3	-0.17	-0.89	0.05	-0.56	0.00	-0.06	0.22	-0.09	-0.37	1															
L4	0.91	0.12	-0.43	0.43	0.86	0.74	0.44	0.69	0.95	-0.30	1														
L5	0.92	0.02	-0.44	0.37	0.89	0.76	0.48	0.70	0.93	-0.19	0.99	1													
SD_{TS}	0.73	-0.17	-0.74	0.51	0.54	0.88	0.54	0.83	0.48	0.09	0.67	0.70	1												
V_{TM}	0.57	0.08	-0.33	0.32	0.47	0.81	0.34	0.70	0.41	-0.06	0.62	0.63	0.73	1											
V_{LP}	0.31	-0.65	-0.33	-0.11	0.33	0.56	0.32	0.58	0.05	0.54	0.20	0.27	0.46	0.44	1										
V_{SP}	0.25	-0.66	-0.32	-0.12	0.28	0.50	0.29	0.51	0.02	0.50	0.16	0.22	0.40	0.36	0.96	1									
LGL	-0.17	-0.89	0.05	-0.56	0.00	-0.06	0.22	-0.09	-0.37	1.00	-0.30	-0.19	0.09	-0.06	0.54	0.50	1								
LAG	0.87	0.06	-0.37	0.34	0.87	0.55	0.36	0.56	0.99	-0.24	0.95	0.95	0.52	0.42	0.13	0.09	-0.24	1							
√PTL	0.76	-0.06	-0.66	0.51	0.56	0.99	0.47	0.97	0.55	-0.08	0.74	0.75	0.89	0.77	0.56	0.50	-0.08	0.56	1						
$PTL^{0.3}$	0.72	-0.07	-0.56	0.42	0.57	1.00	0.47	0.93	0.52	-0.04	0.74	0.76	0.86	0.83	0.56	0.49	-0.04	0.53	0.98	1					
$PTL^{1.5}$	0.72	-0.08	-0.54	0.41	0.57	1.00	0.47	0.92	0.51	-0.04	0.74	0.76	0.86	0.83	0.56	0.49	-0.04	0.53	0.98	1.00	1				
Ln(PTL)	0.75	-0.06	-0.63	0.48	0.56	1.00	0.48	0.96	0.54	-0.07	0.74	0.76	0.89	0.79	0.56	0.50	-0.07	0.56	1.00	0.99	0.99	1			
1/G1	0.71	-0.08	-0.52	0.38	0.57	0.99	0.47	0.91	0.50	-0.03	0.74	0.75	0.85	0.84	0.56	0.49	-0.03	0.52	0.97	1.00	1.00	0.98	1		
√PTL /G1	0.00	-0.97	-0.04	-0.53	0.19	0.09	0.37	0.05	-0.23	0.94	-0.14	-0.04	0.18	-0.01	0.66	0.64	0.94	-0.10	0.07	0.10	0.10	0.08	0.11	1	
1/ PTL	0.40	-0.81	-0.34	-0.18	0.46	0.60	0.60	0.53	0.08	0.72	0.28	0.37	0.61	0.43	0.82	0.77	0.72	0.19	0.58	0.61	0.61	0.59	0.61	0.84	1

The Eqs.4.5 and 4.6 are developed based on the assumption that the drivers perceive the complexity ahead on road at some distance before the point of speed observation. From Eq.4.7 it is observed that the sight distance available at the start of tangent is also having significant effect on operating speed tangent. The Eqs.4.9 to 4.12 are the nonlinear models showing higher R^2 value and having fruitful correlation with geometric variables. The Eq.4.13 shows better R^2 value, but the coefficient of variables (t-test value <2) in the model is not significantly different from zero, hence ln(PTL) i.e logarithemic value of PTL, was not considered further to predict the speed. Even though the variable sight distance satisfies 95% confidence level, when considered along with various combination of PTL, the coefficient of variable SD_{TS} fail to satisfy the t-test condition, but shows higher R^2 value. Similar results were obtained when combination of two or more nonlinear PTL variables is considered. Therefore, such models and the models in Eq.4.14 to Eq.4.16 were removed from the analysis. Among various models developed the model in Eq.4.8 was considered to be the most appropriate model, which satisfies all the conditions considered during the development of speed prediction models. The negative sign of coefficient of 1/ PTL in the model indicates that as the length of tangent section increases, the deduction value from constant decreases, hence, the operating speed increases.

4.5.3 Operating Speed Models at Limit Point

From the correlation matrix given in Table 4.3 and scatter plots shown in Fig.4.5, it is observed that PTL, G1, L1 and LLS are the variables which are likely to influence operating speed at limit point. The variable G1 is highly related to operating speed at limit point than other variables considered. From the Figs.4.5 (a) and (b) it is observed that as PTL and L1 increases, the operating speed at limit point also increases. From these Figures, it is also clear that the influence of PTL is more on operating speed at Limit point than L1; therefore, L1 is not considered for the model development. The distance from limit point to summit point is an indicator of steepness of vertical curvature. If the length LLS is less, it indicates a sharp vertical curve. From the Fig.4.5 (c) it can be observed that as LLS increases, drivers tend to increase the speed in order to overcome the gravitational retardation caused by the gradient of road.

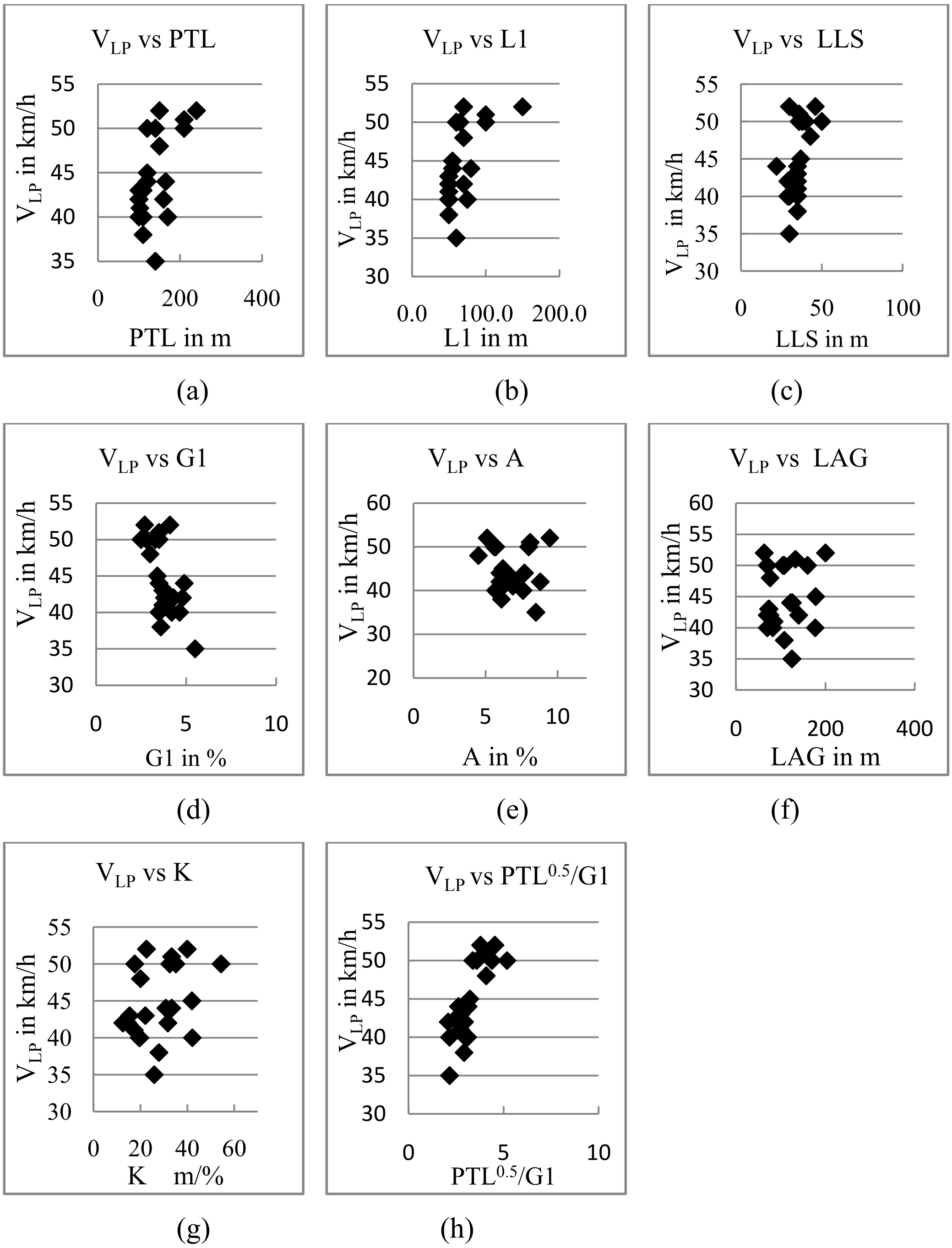


Fig. 4.5 Relationship between operating speed at limit point and geometric variables

From the Fig.4.5 (d) it is observed that as G1 increases, the operating speed at limit point decreases, because the approach gradient forces the driver to reduce the speed of vehicles. In the Figs.4.5 (e) to (g) no clear relations are observed to predict operating speed at limit point, hence the variables A, LAG and K are not considered further. It is also observed from Fig.4.5 (h) that the operating speed at limit point is also better related to nonlinear variable of PTL. With these concepts, initially linear regression analysis was performed to select the variable to be considered for the final model.

The developed operating speed models at limit point are:

$$V_{LP} = 34.936+ 0.070PTL \qquad R^2=0.315 \qquad \text{-- (4.17)}$$

$$V_{LP} = 60.987 -4.347G1 \qquad R^2=0.426 \qquad \text{-- (4.18)}$$

$$V_{LP} = 24.456+1.756PTL^{0.5} \qquad R^2=0.315 \qquad \text{-- (4.19)}$$

$$V_{LP} = 10.479+ 7.825PTL^{0.3} \qquad R^2=0.315 \qquad \text{-- (4.20)}$$

$$V_{LP} = 38.422+0.004\ PTL^{1.5} \qquad R^2=0.313 \qquad \text{-- (4.21)}$$

$$V_{LP}= 55.888-\frac{1467.142}{PTL} \qquad R^2=0.306 \qquad \text{-- (4.22)}$$

$$V_{LP} = 28.15+\frac{59.975}{G1} \qquad R^2=0.432 \qquad \text{-- (4.23)}$$

$$V_{LP} = 29.378+ 0.436LLS \qquad R^2=0.29 \qquad \text{-- (4.24)}$$

$$V_{LP} = 42.856+0.016LAG \qquad R^2=0.018 \qquad \text{-- (4.25)}$$

All the Eqs.4.17 to 4.23 have a logical explanation for the effect of predictor variables on the operating speed at limit point. The model in Eq.4.23 has higher R^2, but the coefficients are not significantly different from zero, hence they are not considered. The Eqs.4.24 and 4.25 does not show good R^2 value compared to other models developed, hence the variables in the model are not considered further.

Comparing the Eq.4.5 with Eq.4.17 and the Eq.4.8 with Eq.4.22, it can be observed that the constant in the models decreases, which indicates deceleration of vehicle speed approaching the limit point where visibility is less. The variables PTL, G1, $PTL^{0.5}$, $PTL^{1.5}$ and 1/PTL are considered further to develop operating speed prediction models at limit point using multi-linear regression technique. The developed models are:

$$V_{LP} = 51.128+ 0.064PTL- 4.116G1 \qquad R^2=0.697 \qquad \text{-- (4.26)}$$

$$V_{LP} = 54.394+ 0.003PTL^{1.5}- 4.135G1 \qquad R^2=0.695 \qquad \text{-- (4.27)}$$

$$V_{LP} = 69.896 - \frac{1315.107}{PTL} - 4.035G1 \qquad R^2=0.669 \qquad \text{-- (4.28)}$$

$$V_{LP} = 41.473 + 1.587PTL^{0.5} - 4.096G1 \qquad R^2=0.690 \qquad \text{-- (4.29)}$$

$$V_{LP} = 28.668 + \frac{4.934PTL^{0.5}}{G1} \qquad R^2=0.668 \qquad \text{-- (4.30)}$$

$$V_{LP} = 33.798 + \frac{0.28\,PTL}{G1} \qquad R^2=0.632 \qquad \text{-- (4.31)}$$

All the Eqs.4.26 to 4.31 have a logical explanation for the effect of predictor variables on the operating speed at limit point. In all equations PTL and G1 are the variables that predict the speed at limit point of vertical curve. Among various models developed, the model in Eq.4.26 presents better logical explanation and higher R^2 value and low PRMSE, hence, the same is considered for the study. The negative sign of coefficient of approach grade, G1 indicates that drivers tend to decrease their speed as G1 increases due to increasing steepness. The positive sign of coefficient PTL means as PTL increases, drivers tend to increase their speed, as the increasing length provides them a platform to accelerate.

Hence, it can be concluded that operating speed at limit point increases as approach tangent length increases and decreases as approach gradient increases. Jessen et al. (2001) also observed that approach tangent grade (G1) is one of influencing variables in predicting operating speed at limited sight distance location on crest vertical curve. Fitzpatrick et al. (2000a) developed speed prediction models using rate of vertical curvature (K), but in this study K does not show any significance to predict the operating speed at limit point of crest vertical curve.

4.5.4 Operating Speed Models at Summit Point

The correlation matrix developed in Table 4.3 and scatter plots in Fig 4.6 show that the variables which influenced the operating speed at limit point, are the variables influencing the operating speed at summit point.

In the Figs.4.6 (a) and (b) it is observed that as PTL and LLS increase, the operating speed at summit point also increases. Longer length of PTL may provide a platform to achieve the desired speed of the vehicle that results in increase in operating speed at tangent point of vertical curve. Also, when the driver travels from limit point to summit point, the sight distance starts to increase; hence the driver accelerates towards approaching the summit point.

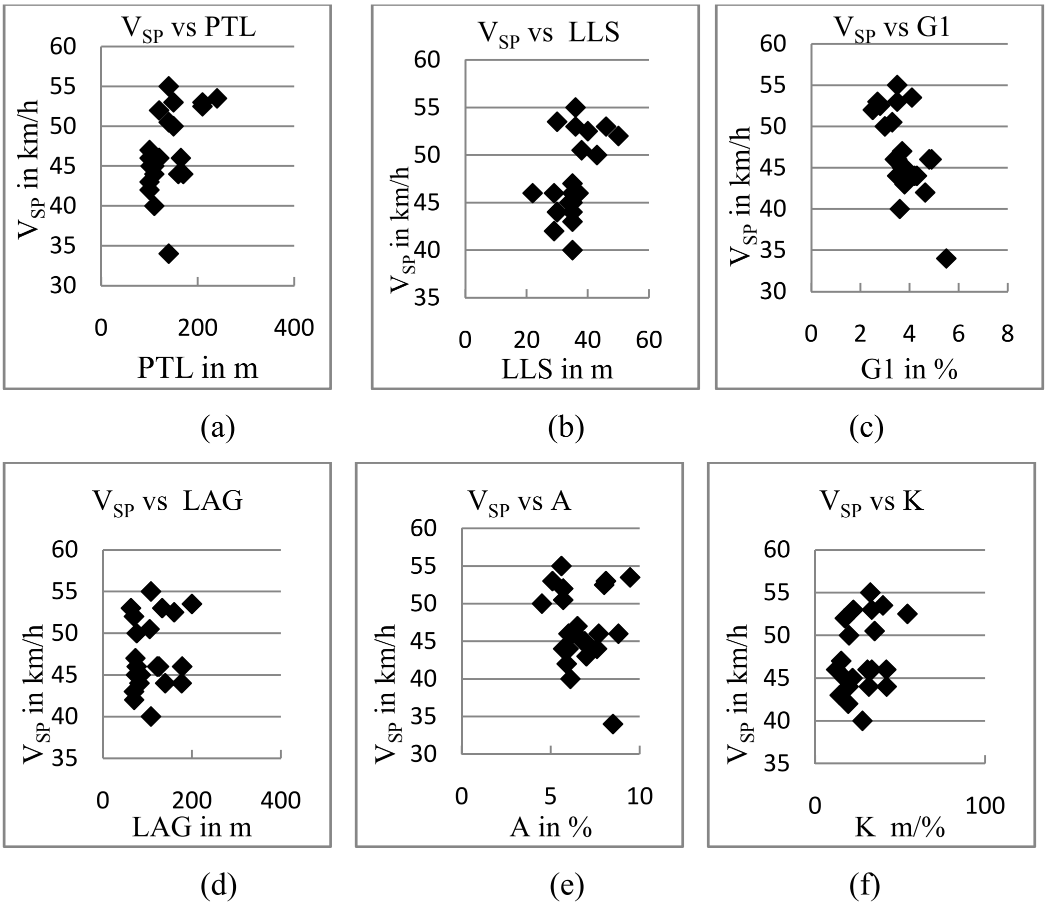


Fig. 4.6 Relationship between operating speed at summit point and geometric variables

Besides, as the length between limit point to summit point (LLS) increases, usually driver decelerates, because of gradient of vertical curve. In the Fig.4.6 (b) it is observed that as LLS increases, the operating speed at limit point also increases. The reason may be that as driver approaches the limit point, psychological tendency of the driver, may force driver to reach the summit point early, when he/she observes a longer distance. Also, the driver accelerating to overcome the gravitational retardation caused due to approaching gradient may be another reason for increase in operating speed at summit point. From the Fig.4.5(c) it is observed that steepness caused due to approach gradient (G1) forces the driver to reduce his/her operating speed at summit point of crest vertical curve. It is clear from the Figs.4.5 (d) to (f) that the variables LAG, A and K do not show any trend to predict the operating speed at summit point of crest vertical curve in intermediate lane rural highways. It is also found from the study that the operating speed at summit point is also shows similar trend with nonlinear variable of PTL and linear variable of PTL considered. With this background, initially linear regression analysis was performed to select the variable to be considered for the final model. The following models are tried to develop the most significant one at summit point:

$$V_{SP} = 38.005 + 0.064PTL \qquad R^2=0.249 \qquad \text{-- (4.32)}$$

$$V_{SP} = 56.890 + \frac{1304.421}{PTL} \qquad R^2=0.226 \qquad \text{-- (4.33)}$$

$$V_{SP} = 64.007 - 4.552G1 \qquad R^2=0.437 \qquad \text{-- (4.34)}$$

$$V_{SP} = 30.339 + \frac{59.975}{G1} \qquad R^2=0.408 \qquad \text{-- (4.35)}$$

$$V_{SP} = 41.162 + 0.003\,PTL^{1.5} \qquad R^2=0.247 \qquad \text{-- (4.36)}$$

$$V_{SP} = 28.532 + 1.562PTL^{0.5} \qquad R^2=0.244 \qquad \text{-- (4.37)}$$

$$V_{SP} = 15.907 + 7.096PTL^{0.3} \qquad R^2=0.242 \qquad \text{-- (4.38)}$$

$$V_{SP} = 78.996 - \frac{139.327}{PTL^{0.3}} \qquad R^2=0.236 \qquad \text{-- (4.39)}$$

$$V_{SP} = 32.36 + 0.415LLS \qquad R^2=0.246 \qquad \text{-- (4.40)}$$

$$V_{SP} = 45.603 + 0.012LAG \qquad R^2=0.009 \qquad \text{-- (4.41)}$$

$$V_{SP} = 66.745 + 0.055PTL - 5.665G1 - 0.178LLS \qquad R^2=0.652 \qquad \text{-- (4.42)}$$

$$V_{SP} = 57.998 - 4.321G1 + 0.003PTL^{1.5} \qquad R^2=0.643 \qquad \text{-- (4.43)}$$

$$V_{SP} = 31.362 + \frac{4.797PTL^{0.5}}{G1} \qquad R^2=0.591 \qquad \text{-- (4.44)}$$

$$V_{SP} = 36.483 + \frac{0.269\,PTL}{G1} \qquad R^2=0.545 \qquad \text{-- (4.45)}$$

$$V_{SP} = 55.09 + 0.058PTL - 4.343G1 \qquad R^2=0.648 \qquad \text{-- (4.46)}$$

Comparing the linear regression models developed in Eq.4.17 with Eq.4.32 and Eq.4.26 with Eq.4.46, it can be observed that the increasing value of constants in the equations indicates the acceleration of vehicles from limit point to summit point of vertical curve. Gradual increase of sight distance from limit point to summit point may be the reason for increase in operating speed at summit point of curve.

All the Eqs.4.32 to 4.46 give a logical explanation for the effect of predictor variables on the operating speed at summit point. Comparing the model in Eq. 4.32 with Eqs. 4.36 to 4.39, higher R^2 is obtained with linear model than nonlinear model with variable PTL. In all models it is clear that positive sign of linear and nonlinear variable of PTL indicates that the driver accelerates the vehicle as preceding tangent length increases. Long straight stretch will reduce the mental stress/workload and give a platform to accelerate the vehicle. This is also another factor which influences the increase of operating speed. The variable in Eq. 4.38 does not satisfy the t-test condition, but when the inverse of variable $PTL^{0.3}$ is considered, it satisfies t-test condition with low R^2 value (one of the examples is given in Eq.4.39). Hence this variable is not considered further.

It is clear from Eqs.4.34 and 4.35 that approaching gradient (G1) is one of the main influencing variables. The variable G1 has higher influence on operating speed at summit point than the variable 1/G1. The negative sign of coefficient of approach grade, G1 implies that as G1 increases, the road becomes steeper and consequently climbing becomes more difficult and hence reduction in speed. Eq.4.40 indicates that the increase of variable LLS has an influence on predicting the operating speed. It is clear from the Eq.4.41 that LAG is not the influencing variable to predict operating speed. Even though the Eq.4.42 shows higher R^2 value, the coefficients of variable LLS fails to satisfy t-test condition.

The nonlinear variable of PTL/G1 gives higher influence (Eq.4.44) than the linear model with variable PTL/G1 (Eq.4.45).The multi regression model in Eq.4.46 shows that the variables PTL and G1 have a higher influence on predicting the speed than the other combinations tried. This model shows better R^2 value and low PRMSE, and hence, is considered to predict the operating speed at summit point of vertical curve. The positive sign of coefficient PTL means that as PTL increases, drivers tend to increase their speed, as the increasing length provides them a platform to accelerate but, the gradient forces them to reduce the speed. Hence it can be concluded that operating speed at summit point increases as approach tangent length increases and decreases as approach gradient increases.

4.5.5 Recommended Operating Speed Models for Crest Vertical Curves

After developing the models at study points, the most significant and logical models are selected in such a way that, in which all the variables satisfy F-test, t-test, and are significant at 95% confidence level. Based on coefficient of regression (R^2) value and PRMSE, most significant operating speed prediction models are considered for crest vertical curves are listed in Table 4.4. The variables in operating speed models presented in Table 4.4 show significance value p<0.0001 and further recommended for consistency evaluation of crest vertical curves.

Table 4.4 Operating speed models for crest vertical curves

Study point at	Model	R^2	R_a^2	F-value	PRMSE	
					Cal. data	Val. data
Tangent	$V_{TM}= 68.451 - \frac{1753.909}{PTL}$ (t= 35.695) (t= -7.211)	0.722	0.708	51.99	3.6	3.3
Limit point	V_{LP}= 51.128+ 0.064PTL-4.116G1 (t=12.68) (t= 4.096) (t= -4.86)	0.697	0.663	21.63	6.4	9.7
Summit point	V_{SP}= 55.09+ 0.058PTL-4.343G1 (t=12.21) (t= 3.31) (t= -4.58)	0.648	0.605	17.11	6.0	9.3

Note: Cal. data = Calibration data
Val. data = Validation data

4.5.6 Validation of Operating Speed Models

The values of the speed predicted by the developed models are need to be checked against the observed values at different study points along the curves. Therefore, a comparison was made between the observed and predicted values of speed for the two data sets. Fig.4.7. show the scatter plots plotted for predicted and observed operating speed values of recommended models at tangent point (Eq.4.8), limit point (Eq.4.26) and summit point (Eq. 4.46) of crest vertical curves.

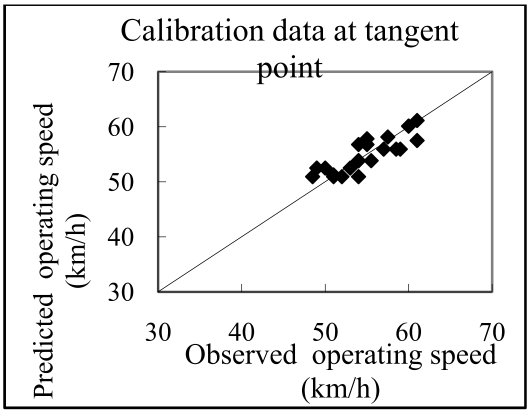
For both sets the Percentage Root Mean Square Error (PRMSE) values were calculated to check the goodness of fit of the models. It is observed from Figs.4.7 (a) to (c) and (e) that both observed and predicted operating speeds at study points are along 45^{o} line drawn showing good fitness. Figs. 4.7(d) and (f) show that both observed and predicted operating speeds of validation data sets at limit point and summit point fall little below the 45^{o} line drawn, but the points are very near to the line which indicates better results. From the scatter plots, one can conclude that the speed prediction models are good enough in predicting the operating speed models at study points.

4.6 DEVELOPMENT OF CONSISTENCY EVALUATION CRITERIA FOR CREST VERTICAL CURVES

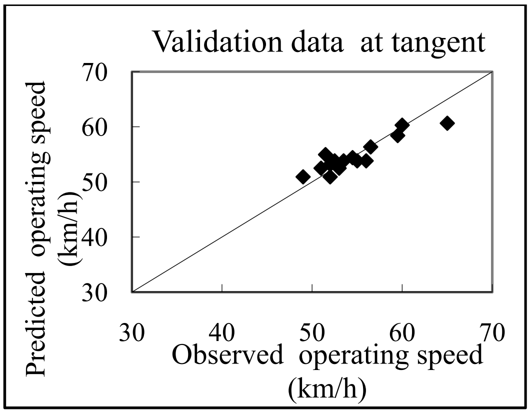
In road sections sharp reduction in speed is very dangerous and hence such sections need to be eliminated while designing the alignment. The consistency in the road section requires gradual speed change from one section to another, so that a driver comfortably controls the vehicle over the sections. Consistency evaluation can be done by considering single element and successive elements of vertical curve. Following are the different methods tried to develop design criteria.

4.6.1 Based on difference between Required and Available length of Crest Vertical curve

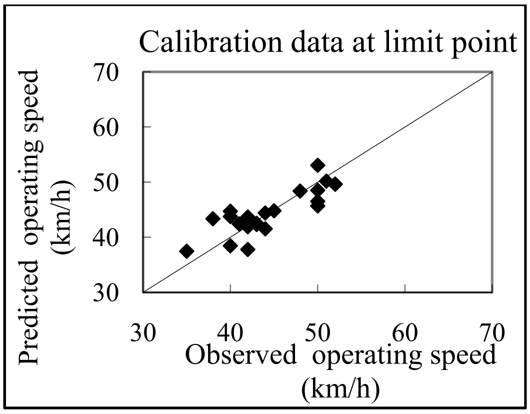
Defference in length of vertical curve was considered to be critical for crest vertical curves. Hence, consistency evaluation is developed relating to available length of vertical curve and required length of crest vertical curve.



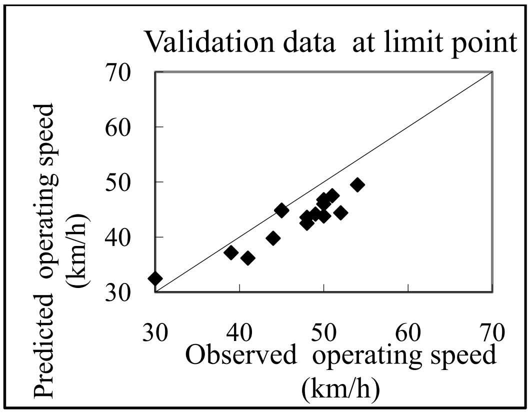
(a)



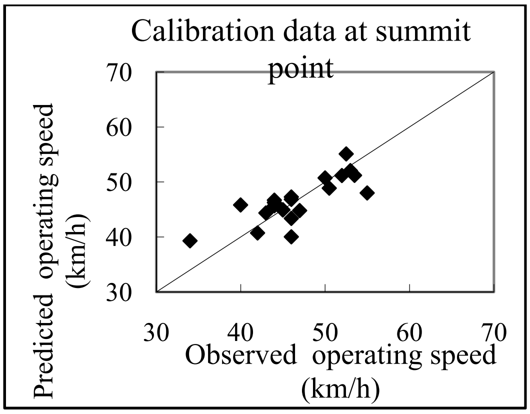
(b)



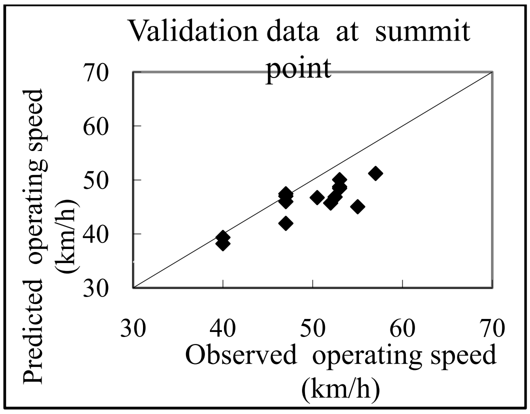
(c)



(d)



(e)



(f)

Fig 4.7 Validation of models developed at study points

Available length was measured from section plan and required length was calculated based on stopping sight distances.

The stopping sight distance was calculated using the formula in Eq. 4.47.

$$SSD = 0.278Vt + \frac{V^2}{254(f \pm 0.01G1)} \quad \text{-- (4.47)}$$

Then length of vertical curve was calculated based on the condition L_V>SSD or L_V<SSD

When,

L_V>SSD

Required length of vertical curve $$RL_V = \frac{A \text{ x } SSD^2}{\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2} \quad \text{-- (4.48)}$$

And when L_V<SSD

Required length of vertical curve $$RL_V = 2SSD - \frac{\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}{A} \quad \text{-- (4.49)}$$

Where,

L_V = length of vertical curve (m)

A = Algebraic difference of grades = [G1-(±G2)]

G_1 = Approach grade (%)

G_2 = Departing grade (%)

SSD= Stopping sight distance (m)

t = reaction time of the driver in seconds (t = 2.5 seconds)

h_1 = height of eye level of driver above road surface (m)

h_2 = height of subject or obstruction above pavement surface (m)

V = Design speed.

A design speed of 80.00 km/h (as per IRC for State Highways in rolling terrain) was considered.

f = Coefficient of longitudinal friction (0.4-0.35).

The defference between curve length (DL_V) is the difference between available length (L_V) and required length of vertical curve (RL_V) as tabulated in Table 4.5. Difference in curve length (DL_V) is related to number of accidents and type of accidents, and safety consistency matrix was developed as shown in Fig. 4.8. The accidents are classified as 0, 1 and >2 number of accidents (Total number of accidents).

Table 4.5 Trial consistency measures – Crest vertical curves

Curve	L_V	RL_V	DL_V	V_d-V_{LP}	V_{Lp}-V_{SP}	V_{TM}-V_{LP}	SD_{TP}-L3	SD_{LP}-L3	Type of Accidents			TA
									FA	GI	SI	
V2	100.0	168.4	-68.4	30.0	-2.0	5.5	20.9	11.0				0
V3	120.0	177.7	-57.7	39.0	-4.0	10.0	26.9	10.6		1		1
V4	110.0	176.4	-66.4	37.0	-2.0	10.0	31.3	14.6				0
V5	200.0	199.9	0.1	36.0	-2.0	10.0	32.3	15.4	1			1
V6	170.0	202.7	-32.7	42.0	-2.0	11.0	23.5	6.1				0
V7	110.0	187.9	-77.9	38.0	-4.0	10.0	33.9	17.6				0
V8	200.0	191.0	9.0	30.0	-0.5	8.5	37.8	22.4				0
V9	190.0	195.6	-5.6	38.0	-2.0	19.0	49.7	16.9		1		1
V10	115.0	158.7	-43.7	28.0	-1.0	2.0	22.0	18.4				0
V11	90.0	146.1	-56.1	32.0	-2.0	7.0	26.6	14.2		2		2
V13	240.0	189.9	50.1	40.0	-4.0	10.0	25.3	9.2			1	1
V14	237.0	246.9	-9.9	36.0	-2.0	11.0	46.1	27.7		1		1
V15	378.0	310.0	68.0	28.0	-1.5	9.0	49.9	33.4		1	1	2
V16	270.0	269.9	0.1	29.0	-2.0	9.0	42.5	26.0				0
V17	435.0	271.9	163.1	30.0	-2.5	10.0	39.2	20.8				0
V18	260.0	207.2	52.8	35.0	-1.0	9.0	30.4	15.0		1		1
V19	115.0	163.8	-48.8	40.0	-2.0	14.0	37.5	14.7				0
V20	102.0	178.3	-76.3	38.0	-1.0	6.5	22.6	12.1				0
V21	182.0	186.6	-4.6	30.0	-5.0	7.0	36.8	24.3				0
V22	100.0	173.8	-73.8	37.0	-4.0	8.0	26.9	13.7				0
V23	150.0	248.4	-98.4	40.0	-4.0	17.5	43.2	13.9		2		2
V24	220.0	268.3	-48.3	45.0	1.0	24.0	44.8	6.2		2		2
V25	215.0	232.0	-17.0	32.0	1.0	4.5	29.5	21.8				0
V26	165.0	210.7	-45.7	30.0	-2.0	9.5	51.6	34.7				0
V27	140.0	221.0	-81.0	39.0	1.0	12.5	35.0	14.8				0
V28	240.0	242.2	-2.2	32.0	-7.0	1.0	23.5	21.8				0
V29	200.0	206.8	-6.8	29.0	-2.0	0.5	13.3	12.5				0
V30	130.0	175.1	-45.1	35.0	-2.0	11.5	31.7	11.8				0
V31	160.0	196.1	-36.1	30.0	-3.0	2.0	29.3	25.8				0
V32	240.0	193.8	46.2	31.0	-1.5	2.0	23.1	19.7				0

Table 4.5 Trial consistency measures – Crest vertical curves											(Continued…)
V33	120.0	174.4	-54.4	28.0	-0.5	4.0	35.0	27.8			0
V34	240.0	242.4	-2.4	41.0	-1.0	14.0	39.4	17.0		1		1
V35	335.0	230.8	104.2	30.0	-3.0	10.0	45.7	27.7			1	1
V36	185.0	168.1	16.9	26.0	-3.0	11.0	52.9	32.0		1		1
V37	285.0	241.8	43.2	50.0	1.0	25.0	36.0	1.6		2	1	3
V38	210.0	241.8	-31.8	36.0	-3.0	10.5	27.3	9.8				0
V39	235.0	268.1	-33.1	35.0	-2.0	7.0	24.9	13.1		1		1
V40	140.0	300.7	94.3	45.0	-1.0	27.0	25.9	-18.6				0

The number of curves falling under each category was counted. The number of curves falling under each category was counted. In this method maximum results of 34.2 % i.e. 13 vertical curves out of 38 curves (selected for analysis) was obtained with difference range of curve length at<30 m, 30-50 m and >50 m.

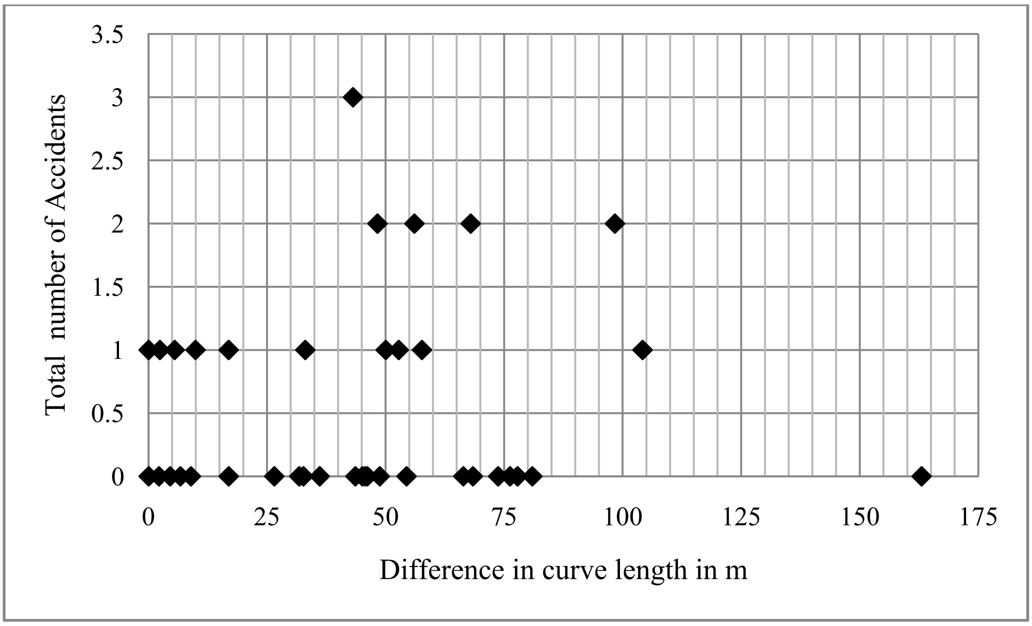


Fig 4.8 Safety consistency matrix for DL_V

4.6.2 Based on Difference between Design Speed and Operating Speed at Limit Point

Individual drivers vary their operating speeds to adjust to features encountered along the road which resuts in, greater and more frequent the speed variations. These speed variations lead to higher probability of an occurrence of collision (Hassen 2004). Hence, design consistency evaluation of vertical curves was also done by finding the relationship between the total number of accidents and type of accident with the difference between design speed and operating speed at limit point (V_d-V_{LP}). The design speed of 80 km/h was used (IRC recommended value for state highways passing in a rolling terrain). Safety consistency matrix was plotted using the difference between design speed and operating speed at limit point and with accident history.The number of curves falls under different ranges of V_d-V_{LP} with number of accidents are counted.

In this method 47.3% (18 out of 38) vertical curves, agrees with the classification range of V_d-V_{LP} at <36 km/h, 36-44 km/h and >44 km/h with total number of accidents (classified as 0, 1 and >2 number) as shown in Fig.4.9.

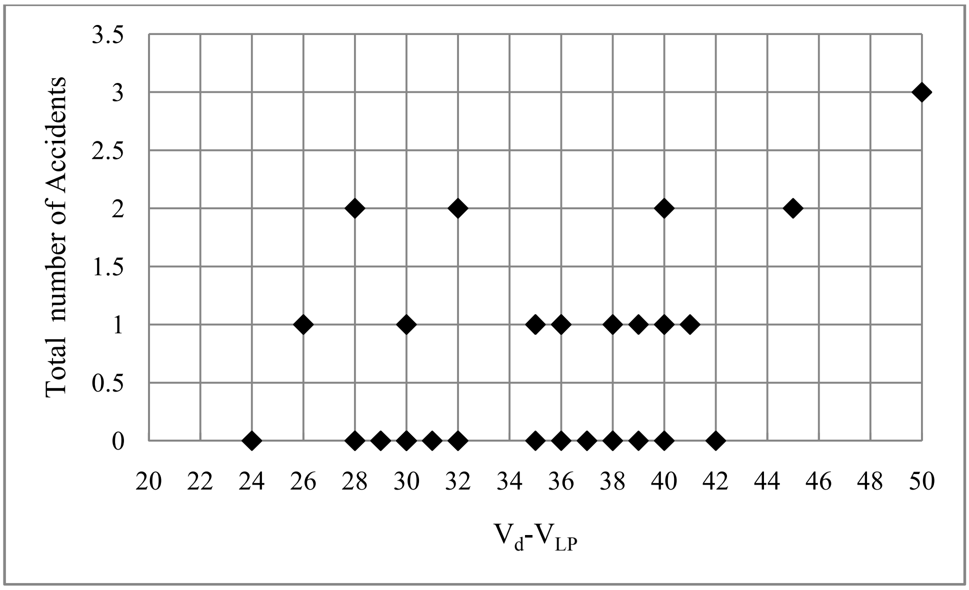


Fig 4.9 Safety consistency matrix for V_d-V_{LP}

4.6.3 Based on difference between sight distance at tangent (SD_{TP}) and L3

Sufficient sight distance should be available throughout the crest vertical curves to avoid inconsistency or hazardous situations (Wooldridge et al. 2003). At least, throughout vertical curves the sight distance is available in such a way that the driver can stop safely, travelling at operating speed at tangent (V_{TM}) of vertical curve. Because of gradient of the crest vertical curve at limit point the available sight distance reduces to a minimum. In this study it is hypothesised that higher deficiency in sight distance required based on tangent speed (V_{TM}) and L3 (sight distance available at limit point) may represent design in consistency. Therefore, sight distance required throughout the vertical curve is calculated using the Eq.4.43. Then the difference between calculated sight distance at tangent point (SD_{TP}) and L3 is calculated and the values are given in Table 4.5.

To find the best classification, different trials of differences of SD_{TP} and L3 are compared with total number of accidents observed. From the Fig.4.10 it can be observed that a maximum of 34.2 % (13 out of 38 curves) vertical curves matches with range of classification 25 m, 25-35 m and >35 m.

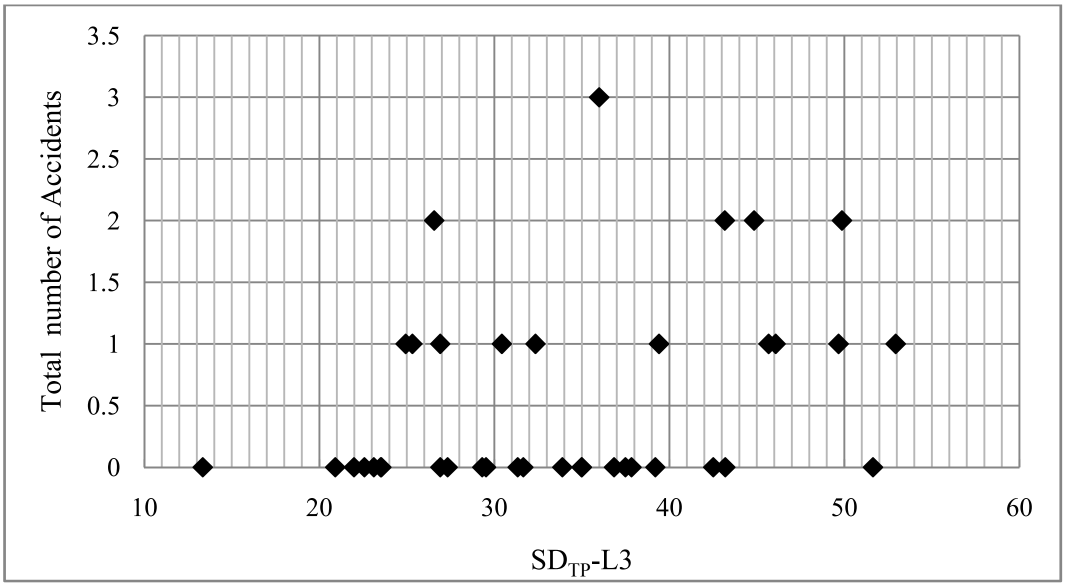


Fig 4.10 Safety consistency matrix for SD_{TP}-L3

4.6.4 Based on Difference between Required Sight Distance at Limit Point (SD_{LP}) and L3

For the safe operation of vehicle throughout the length of crest vertical curve, at least available sight distance should be equal to the sight distance to be required for the vehicle travelling at operating speed V_{LP} . Therefore, it was assumed that, the difference in available sight distance at limit point (L3) and required sight distance (SD_{LP}) based on V_{LP} may represent the design consistency. The required sight distance was calculated using the Eq.4.43 by considering operating speed at limit of vertical curve. The calculated differences of SD_{LP} and L3 are given in Table 4.5. It is observed from the safety consistency matrix in Fig. 4.11 that it is not possible to find the threshold values of difference of SD_{LP} and L3 based on accident history. Hence it can be concluded that this method cannot be used for consistency evaluation of crest vertical curves.

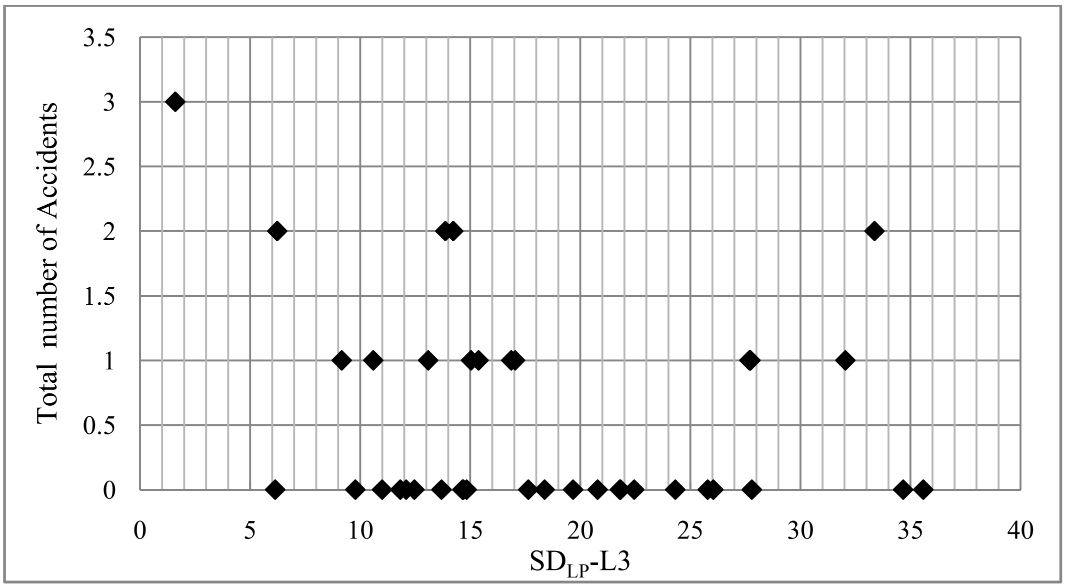


Fig. 4.11 Safety consistency matrix for SD_{LP}-L3

4.6.5 Based on Difference between Operating Speeds at Limit Point and Summit Point

Gradient and restricted sight distance at limit point forces a driver to reduce his/her speed and as the vehicle approaches the summit point with increase in visibility the driver increases the vehicle speed. It is hypothesised that if this increase in speed is more than certain limit, it may lead to dangerous situations. Therefore, the difference between operating speed at limit point and operating speed at summit point (V_{LP}-V_{SP}) is considered to develop consistency evaluation criteria.

Table 4.5 shows the calculated values of V_{LP}-$V_{SP.}$ From the safety consistency matrix developed for V_{LP}-V_{SP} shown in Fig. 4.12, it is observed that it is not possible to develop the design consistency criteria based on accident history.

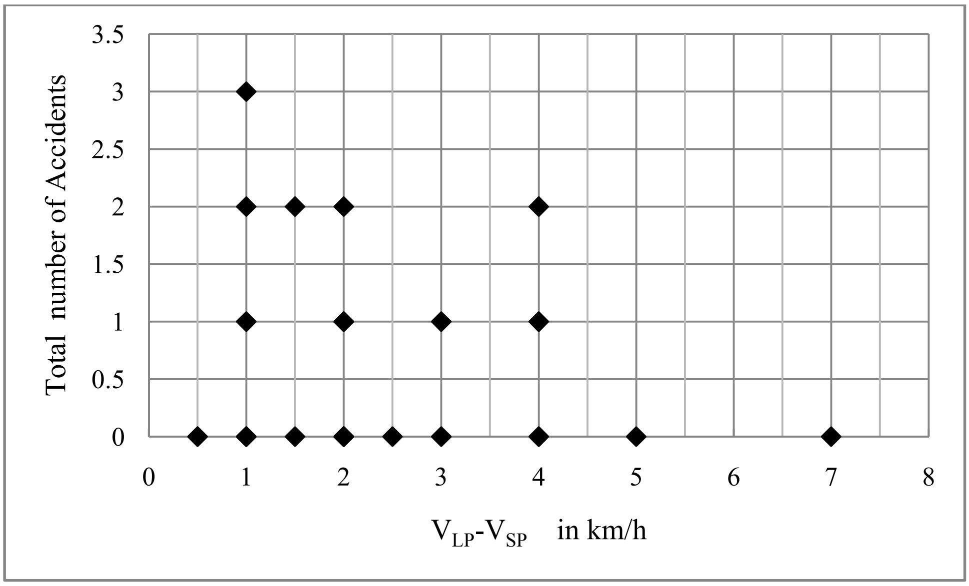


Fig.4.12 Safety consistency matrix for V_{LP}-V_{SP}

4.6.6 Based on Difference between Operating Speeds at Tangent and Limit Points

Drivers travelling substantially less or more than the average traffic speed between successive elements of curve have a higher risk of being involved in collisions (Geometric design guide for Canadian roads 1999). Therefore, the difference between operating speeds at one element to the other element is a good measure of consistency evaluation. The calculated values of difference in operating speed between tangent point and limit point (V_{TM}-V_{LP}) are given in Table 4.5. Fig 4.13 shows the safety consistency matrix developed for V_{TM}-$V_{LP,}$ which gives better idea of the consistency classification. A numbers of trials were performed with different ranges of difference in operating speeds between tangent and limit point and compared with accident history. Among different sets of speed ranges tried, 0-10 km/h, 10-20 km/h, and > 20 km/h gave good results with number of accidents and based on this 60.5% (23 out of 38 curves) curves were correctly classified.

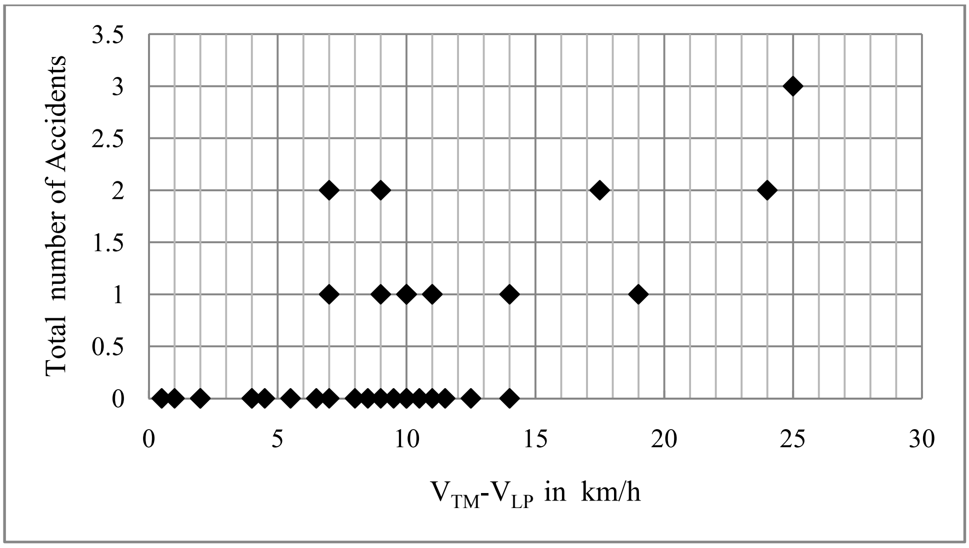


Fig 4.13 Safety consistency matrix for V_{TM}-V_{LP}

4.6.7 The Developed Consistency Evaluation Criteria for Vertical Crest Curves

Among various consistency evaluation criteria developed, it is necessary to identify the criteria to be considered for consistency evaluation of vertical summit curves in intermediate lane rural highways. The consistency evaluation criteria developed in section 4.6.1 to section 4.6.4 are for single element of vertical curve and section 4.6.5 and section 4.6.6 for successive elements of vertical curve. In this study, consistency evaluation criteria for crest vertical curves in intermediate lane rural highways are considered based on the best results obtained for single element and successive elements as given in Table 4.6 and Table 4.7 respectively.

Table 4.6 Consistency evaluation criterion for crest vertical curve -Single element

Consistency measure	Criterion	Evaluation
V_d-V_{LP}	<36 km/h	Good
	36-44 km/h	Satisfactory
	>44 km/h	Poor

The design consistency evaluation criteria considered for crest vertical curves of intermediate lane rural highways given in Table 4.7 is similar to Lamm's criteria (1988,1995) developed for successive elements of horizontal curves of two lane rural highways.

Table 4.7 Consistency evaluation criterion for crest vertical curve -Successive element

Criterion	Consistency measure	Evaluation
$\Delta V = V_{TM}\text{-}V_{LP}$	<10 km/h	Good
$\Delta V = V_{TM}\text{-}V_{LP}$	10-20 km/h	Satisfactory
$\Delta V = V_{TM}\text{-}V_{LP}$	> 20 km/h	Poor

4.7 SUMMARY

Operating speed concept was the most significant method to evaluate the consistency of crest vertical curves in intermediate lane rural highways. The chapter explains the geometric, speed, and accident data collection for vertical curves. The significant variables that influence the operating speed at tangent point, limit point and summit point are identified. The operating speed at tangent point depends on preceding tangent length. The operating speed at limit point and summit point depends on approach grade and preceding tangent length. This chapter focuses on the development of evaluation criteria for single and successive element of vertical summit curves.

CHAPTER 5

CONSISTENCY EVALUATION BY ALIGNMENT INDICES

5.1 INTRODUCTION

Highway collisions are a major source of both social and economic losses. Improvement of highway safety performance is the concept of highway design consistency. Alignment indices are the alternate method for evaluating the design consistency of a roadway. This chapter provides information on classification of alignment indices and data required for the consistency evaluation of intermediate lane rural highways. The data collection methodology and calculation method of selected various alignment indices are also discussed in detail. An attempt has been made relating the alignment indices to the total accidents. This chapter also presents the design consistency evaluation criteria developed for stretches using alignment indices.

5.2 ALIGNMENT INDICES

Alignment indices are quantitative measures of the general character of a roadway segment's alignment. Problems with geometric inconsistencies will arise when the general character of alignment changes between segments of a roadway. A common example is where the terrain transitions from level to rolling or mountainous, and the alignment correspondingly changes from gentle to more severe. The average of the geometric parameters along a roadway segment can indicate the general character of the road. The variation in the alignment indices can be better interpreted by comparing the individual alignment feature with the average feature of a roadway.An individual feature that has a value dissimilar to that of the average of the roadway can possibly indicate that there is some inconsistency between that feature and the general alignment of the roadway.

Alignment indices have several advantages for use in design consistency evaluations (Anderson et al. 1999). First, they are easy for designers to use, understand, and explain. Second, alignment indices are a function of horizontal and/ or vertical alignment elements. Therefore, they would provide a mechanism for quantitative assessment of successive elements from a system-wide perspective, which is a fundamental motivation of design consistency research. Third, alignment indices attempt to quantify the interaction between horizontal and vertical alignments, a design strategy that is currently missing from design policy. Fitzpatrick et al. (2000b) concluded that alignment indices provide more information related to the consistency of a section of roadway than as an individual measure.

5.3 CLASSIFICATION OF ALIGNMENT INDICES

Fitzpatrick et al. (2000a) classified the alignment indices that were used for consistency evaluation of alignment sections as horizontal alignment indices, vertical alignment indices, and composite alignment indices. The design inconsistencies in the horizontal alignment can best be identified using horizontal alignment indices, and inconsistencies in the vertical alignment can at best be identified using vertical alignment indices. The composite alignment indices provide information on design inconsistencies that exist on both horizontal and vertical elements along a roadway. Fitzpatrick et al. (2000a) and Fitzpatrick et al. (2000b) identified some of the proposed alignment indices and grouped them according to their relation with geometry, as explained below:

- **Horizontal alignment indices**
  - Angular change in direction
    - Average deflection angle per length (Bendiness or Curvature Change Rate (CCR)
    - Average deflection angle per curve
    - Average degree of curvature per length
    - Average degree of curvature per curve
    - Ratio of curve length to total roadway length

- Radii measures
    - Average radius
    - Ratio of average radius to minimum radius
  - Ratio of maximum radius to minimum radius
    - Ratio of an individual curve radius to average radius
- Tangent length
    - Average tangent length
    - Ratio of an individual tangent length / average tangent length
- Sight distance
    - Harmonic mean visibility

- **Vertical alignment indices**
  - Angular change in direction
    - Algebraic difference in grades / length or vertical CCR (percent/km) (degrees/km)
    - Average rate of vertical curvature
  - Elevation measures
    - Average gradient
  - Sight distance
    - Sight distance
- **Composite alignment indices**
  - Average highway speed
  - Angular change in direction or combination CCR)

5.3.1 Horizontal Alignment Indices

The horizontal alignment indices recommended are indicators of the amount of curvature of the road and how winding it is. The horizontal angular change in direction indices are the representative of the general character of the alignment of the roadway. A large value for these indices indicates that the road either contains a large number of curves or there are long or sharp curves in that section. It was expected that an increase in the value of these indices would decrease the desired speeds of motorists. Correspondingly, the ratio of curve length to total roadway length also provides a good indication of the character of the alignment. This alignment index provides information on proportion of the roadway that is on curved sections. As the road includes more curves or longer curves, this proportion increases, and the speeds at which motorists drive would be expected to decrease.

The horizontal indices i.e. the ratio of the average to the minimum radius, maximum radii to the minimum radius, and an individual curve radius to average radius provides an indication of design consistency (the closer the value is to 1, the more consistent the design) of individual curves. The average radius expresses what motorists typically encounter on curved sections of the road. A large average radius would indicate curves that are typically not very sharp. Therefore, it is expected that higher speeds would exist for these values compared to smaller average radius values.

The average tangent length indicates the length of tangent that is typically available to motorists between curved sections of the roadway. A large value for this index would indicate that the road has tangent sections that are typically long; therefore, motorists' speeds would be expected to be higher than for roads with a smaller value. The ratio of an individual tangent to average tangent length provides an indication of design consistency (the closer the value is to 1, the more consistent the design) of individual tangent. The sight distance based alignment index is the non-geometry-based index and is measured directly on the field. The amount of sight distance available to motorists is based on subjective parameters, such as driver eye height and object height.

5.3.2 Vertical Alignment Indices

The algebraic difference in grade per length measures the angular change in the vertical direction of a road in the same way as CCR measures the angular change in the horizontal direction. Therefore, this index was renamed the vertical CCR, and the units for this alignment index were converted to match the units of the horizontal CCR. A large vertical CCR value would indicate the existence of many changes in the vertical direction of a roadway or the presence of steep grades.

The average rate of vertical curvature provides a measure of the sharpness of the vertical curves of a road, which resembles the average radius alignment index in the horizontal plane. A large value for this index would indicate that motorists are provided with more sight distance when travelling on the vertical curves of the roadway. Therefore, the effects of a steep grade, if present, may be reduced and the speeds of the motorists may increase as a result. The average gradient index represents the absolute change in vertical direction along a roadway. As the average gradient increases in value, this indicates that either there is typically a large change in elevation between the vertical points of intersections or that the vertical alignment is hilly. In vertical sight distance alignment index, the sight distance available to motorists is not based on the geometric parameters of the road; it uses subjective parameters such as driver eye height and object height.

5.3.3 Composite alignment indices

Of the two composite alignment indices identified, the average highway speed is based on the design speed of the individual alignment elements of the roadway. The composite angular change in direction combines the horizontal and vertical angular change in direction of the roadway. This index was termed the combination CCR because it was computed by simply adding the horizontal CCR value and the vertical CCR value. As the value of the combination CCR increased, it indicated more angular changes in either the horizontal and/or vertical direction of the roadway.

5.4 DATA REQUIREMENT

Cafiso et al. (2005) confirmed from the study that a coordinate sequence of curves does not produce an unexpected driving event even if short bending radii are adopted. Geometric inconsistency produced by a sharp curve following a long tangent produces tense driving behaviour. Driving inconsistencies are due to local maximum curvatures of the vehicle path higher than those required by horizontal alignment. These manoeuvres are caused by a driver's need to suddenly correct his or her driving behaviour due to an unexpected alignment and can produce a dangerous situation. Polus (1980) proposed to correlate highway safety with design elements and, therefore, with consistency, because the drivers tended to build up an expectation of what the upcoming roadway would be like, based on their immediate previous driving experience.

Alignment indices provide a mechanism for quantitative assessment of successive elements from a system-wide perspective, which is a fundamental motivation of design consistency research. Also, they attempt to quantify the interaction between horizontal and vertical alignments, which is missing from current design policy (Fitzpatrick et al. 2000a). Therefore, in this study, to find the relationship between highway geometric elements and accidents, alignment indices are selected as a design consistency measure. The initial step of this consistency evaluation is to select the stretches and data required for the study. Alignment indices are the function of the dimensions of horizontal and/or vertical alignment elements. Therefore, the basic data requirements for the consistency evaluation by alignment indices of any road stretch are: 1) Geometrics and 2) Accident. The geometric data include the details, such as radius and length of the curve, degree of curvature, deflection angle, gradients, and length of tangent of the curve. Accident data include the type and location of accident.

5.5 DATA COLLECTION

The intermediate lane rural highway sections selected for consistency evaluation of horizontal and vertical curves are considered further for consistency evaluation by alignment indices. The selected 57 sections consist of 178 horizontal curves with more than 100 m tangent length, 40 horizontal curves with tangent length of less than 100 m (this also includes nine other horizontal curves which are not considered for consistency evaluation during speed consideration in the selected sections), 76 combined curves i.e. horizontal curves with gradient more than ±2% and 40 vertical summit curves. Horizontal feature of combined curve is taken into account to calculate horizontal alignment indices and vertical feature of it is considered to calculate vertical alignment indices. In the two sections/stretches only the details of vertical curves were extracted; therefore, these stretches/sections (Nos. 11 and 12) are not considered for this consistency evaluation. Finally, in this study, 55 sections are considered for the consistency evaluation by alignment indices.

5.5.1 Geometric Data

The geometric data required for this study include information about the horizontal and vertical elements that exists along selected sections. Surveying was conducted to obtain the geometric details of the study sites. From the survey details, plot of plan and profile were generated.The length of each section selected at different highways varies between 0.4 km to 4.6 km. From the plan of the selected section, the details like radius of the curve (R_c), length of the curve (L_H), degree of curvature (D), and deflection angle (Δ) are extracted, and from the profile of section, the collected details are approach gradient (G1%), departure gradient (G2%), length of vertical curve (L_V) and length of tangent.The estimation of all these parameters is explained in the data collection part of Chapter 3 in section 3.4 and Chapter 4 in section 4.4.

5.5.1.1 Degree of curvature

The sharpness of the curve is designated either by its radius or by its degree of curvature. According to the arc definition generally used in highway practice, the degree of curve is defined as the central angle of the curve that is subtended by an arc of 30 m length.

$$\text{Degree of curvature } D \text{ (in deg)} = 1718.9/R_c \qquad \text{-- (5.1)}$$

Where,

R_c = Radius of curve in m

A sharp curve has a large degree of curve whereas a flat curve has a small degree of curve.

5.5.2 Accident Data

Accident data was collected from First Information Reports of accidents available at various police stations of Dakshina Kannada district. The accident data collection methodology and identification of accident spots are explained in the section 3.6,Chapter 3. The summary of geometric and accident details collected for this study either on the field or from alignment drawings is presented in Table 5.1.

Table 5.1 Summary of collected details

	Minimum	Maximum
Radius of curve (m)	26.5	865
Curve length (m)	30	435
Deflection angle (deg)	6	159
Degree of Curvature(deg)	2	65
Preceding Tangent Length(m)	0	423
Approach Gradient, G1 (%)	0.5	6.4
Departure Gradient, G2 (%)	0.0	6.1
Number of Fatal Accidents (FA)	0	2
Number of Grievous Injury Accidents (GI)	0	5
Number of Simple Injury Accidents (SI)	0	3
Total number of Accidents (TA)	0	6

5.6 METHODOLOGY ADOPTED FOR SELECTION OF ALIGNMENT INDICES

As part of this evaluation, some of the proposed alignment indices explained in section 5.3 were computed for a small sample of highway sections. After examining these results and the proposed alignment indices more closely, it was determined that some of the indices may be more worthwhile for use in estimating the accident frequencies than others. Two other criteria used in determining whether the indices should be further evaluated in this study were:

• Alignment indices had to be strictly a function of the geometry of a roadway.

• There had to be a reasonable hypothesised relationship between the alignment index and the accident frequencies.

The selection of indices was also considered based on results of studies conducted by Polus and Dagan (1987), Anderson et al. (1999), and Fitzpatrick et al. (2000a). Therefore, those indices that were thought to be better measures were calculated in this research while the others were eliminated from consideration, as explained in section 5.7.

5.7 ESTIMATION OF ALIGNMENT INDICES

The initial step of consistency evaluation by alignment indices is to estimate the horizontal alignment indices, vertical alignment indices, and composite alignment indices considered for this study. The formulae used for the calculation of alignment indices are given in Eqs.5.2 to 5.12.

5.7.1 Horizontal Alignment indices

5.7.1.1 Curvature change rate

$$\text{Curvature Change Rate - CCR (deg/km)} = \frac{\sum \Delta i}{\sum Li} \qquad \text{-- (5.2)}$$

Where,

Δi = deflection angle (deg)

Li = length of section (km)

Example for stretch 1: CCR $= \frac{\sum \Delta i}{\sum Li}$

$$= \frac{40}{1} + \frac{48}{1} + \frac{60}{1} + \frac{61}{1}$$

$$= 209.0 \text{ deg/km}$$

5.7.1.2 Degree of curvature

Degree of Curvature - DC (deg/km) = $\frac{\sum DCi}{\sum Li}$ -- (5.3)

Where,

DCi = degree of curvature (deg)

Li = length of section (km)

Example for stretch 1: DC = $\frac{7.8}{1}+\frac{8.6}{1}+\frac{9.5}{1}+\frac{10.6}{1}$

= 36.59 deg/km

5.7.1.3 Curve length: Roadway length

The ratio of curve length to roadway length provides only an indication of the general character of the roadway. This alignment index increases as the road includes more curves or longer curves.

Curve Length: Roadway Length - CL: RL= $\frac{\sum CLi}{\sum Li}$ -- (5.4)

Where,

CLi = curve length (m)

Li = length of section (m)

Example for stretch 1: CL: RL = $\frac{153}{1000}+\frac{168}{1000}+\frac{190}{1000}+\frac{173}{1000}$

= 0.7

5. 7.1.4 Average radius

The average radius expresses the sharpness of the curves that motorists typically encounter on a given section of the roadway. A large average radius would indicate curves that are typically not very sharp. A small average radius indicates that the curves on a roadway section are quite sharp. The average radius for a roadway section is defined as:

$$\text{Average Radius - AR (m)} = \frac{\sum R_c i}{n} \quad \text{-- (5.5)}$$

Where,

$R_c i$ = radius of curve (m)

n = number of curves within section

$$\text{Example for stretch 1: AR} = \frac{219+199+181+162}{4}$$

$$= 190.3 \text{ m}$$

5.7.1.5 Average tangent length

The average tangent length along a roadway can be determined by computing the sum of the individual tangent length divided by number of tangents in the section. For each roadway section, the average tangent length of the roadway section is computed as:

$$\text{Average Tangent Length- ATL (m)} = \frac{\sum TLi}{n} \quad \text{-- (5.6)}$$

Where,

TLi = tangent length (m)

n = number of tangents within section

$$\text{Example for stretch 1: ATL} = \frac{110+0+0+112+94}{5}$$

$$= 63.2 \text{ m}$$

5.7.1.6 Maximum radius/ minimum radius

The range of radii along a roadway can be determined by computing the ratio of the maximum radius to minimum radius. A section composed of curves of approximately the same radius might be considered a consistent design, even if that curve radius was quite sharp, while an otherwise similar section with a broad range of curve radii might be considered an inconsistent design. For each roadway section, the ratio of the maximum radius to the minimum radius of the roadway section is computed as:

$$RR = R_{max}/R_{min} \qquad \text{-- (5.7)}$$

Where,

RR = ratio of maximum radius to minimum radius,

R_{max}= maximum radius for any curve on the roadway section (m)

R_{min}= minimum radius for any curve on the roadway section (m)

Example for stretch 1: $RR = \dfrac{219}{162}$

$= 1.35$

5.7.1.7 Average radius/ minimum radius

If the minimum radius is much different than the average radius, the curve with the minimum radius may be the inconsistency.

$$AR/\ R_{min} = \frac{\text{Average radius}}{\text{Minimum radius}} \qquad \text{-- (5.8)}$$

Example for stretch 1: $AR/\ R_{min} = \dfrac{190.25}{162}$

$= 1.174$

5.7.1.8 Radius/average radius

A section composed of curves of approximately the same radius might be considered a consistent design, even if that curve radius was quite sharp, while an otherwise similar section with a broad range of curve radii might be considered an inconsistent design.

$$R_c/AR = \frac{\text{Radius}}{\text{Average radius}} \qquad \text{-- (5.9)}$$

Example in stretch 1, for H1 curve (i.e. for individual curve):

$$R_c/AR = \frac{219}{190.25}$$

$$= 1.15$$

5.7.2 Vertical alignment indices

5.7.2 .1 Vertical CCR

$$\text{Vertical CCR - VCCR (deg/km)} = \frac{\sum Ai}{\sum Li} \qquad \text{- (5.10)}$$

Where,

Ai = absolute difference in grades (deg)

Li = length of section (km)

Example for stretch 6:

$$\text{VCCR (percent/km)} = \frac{5.7}{1.9} + \frac{6.6}{1.9} + \frac{6.9}{1.9} + \frac{6.0}{1.9} + \frac{2.9}{1.9}$$

$$\text{i.e. VCCR} = \frac{3.26}{1.9} + \frac{3.77}{1.9} + \frac{3.95}{1.9} + \frac{3.43}{1.9} + \frac{1.66}{1.9}$$

$$= 8.47 \text{ deg/km}$$

5.7.2.2 Average rate of vertical curvature

The average rate of vertical curvature provides an indication of the amount of change in the vertical alignment for a roadway. The assumption behind this alignment index is that the amount of hilliness on an alignment can have an effect on speeds and accidents. The average rate of vertical curvature is defined as:

$$\text{Average Rate of Vertical Curvature - AVC (m/ \%)} = \frac{\dfrac{\sum Li}{|A|}}{n} \qquad \text{-- (5.11)}$$

Where,

L= length of the vertical curve on the roadway section (m),

A= algebraic difference of grade (%), and

n = number of vertical curves with in a section.

Example for stretch 6:

$$\text{AVC} = \frac{\left(\frac{100}{5.7}+\frac{150}{6.6}+\frac{120}{6.9}+\frac{100}{6.0}+\frac{80}{2.9}\right)}{5}$$

$$= 20.4 \text{ m/\%}$$

5.7.3 Composite alignment indices

A composite alignment index was also calculated to determine the design consistency of individual features along a roadway.

5.7.3.1 Combination CCR

$$\text{Combination CCR-COMBO (deg/km)} = \frac{\sum \Delta i}{Li} + \frac{\sum Ai}{Li} \qquad \text{-- (5.12)}$$

Where:

Δi = deflection angle (deg)

Ai = absolute difference in grades (deg)

Li = length of section (km)

Example for stretch 6:

$$\text{COMBO} = \left[\frac{99}{1.9}+\frac{75}{1.9}+\frac{57}{1.9}+\frac{59}{1.9}+\frac{86}{1.9}+\frac{101}{1.9}+\frac{65}{1.9}+\frac{40}{1.9}+\frac{50}{1.9}\right]+$$

$$\left[\frac{3.26}{1.9}+\frac{3.77}{1.9}+\frac{3.95}{1.9}+\frac{3.43}{1.9}+\frac{1.66}{1.9}\right]$$

$$= 340.5 \text{ deg/km}$$

Table 5.2 shows the geometric details of the curves, identified accident details in three years and alignment indices calculated for individual features in the selected ten samples of stretches/sections. The same details of other stretches (13-57) are given in Appendix 3, Table A 3.1.

Notations used for the study curves are as follows:

H =Horizontal curves with tangent length more than 100 m with gradient ±2%

AH= Horizontal curves with tangent length less than 100 m with gradient ±2%

CC= Combined curves i.e, horizontal curves with gradient more than 2%

V= Vertical curves

5.8 METHODOLOGY ADOPTED FOR CONSISTENCY EVAUATION

The main premise for using alignment indices as design consistency measures is that geometric inconsistencies will result when the general character of all alignments changes significantly.The alignment indices were calculated for each kilometre of road length and are presented in Table 5.3. The length of the stretches considered for this study varied from one another. Therefore, to make uniformity in the data set to be used for consistency evaluation, accident data is considered as annual accidents/km, which is calculated as:

$$\text{Annual accidents/km} = \frac{\text{Observed total accidents in three years}}{\text{Observed period x Length of stretch}} \quad \text{--(5.13)}$$

Then the alignment indices are related with annual accidents/km for consistency evaluation. If there exists a large increment or decrement in these values among the section, then there exists inconsistency in the geometric design.

5.9 CONSISTENCY EVALUATION OF INDIVIDUAL ALIGNMENT FEATURE BY ALIGNMENT INDICES

To interpret the variation in the general character of the individual curves of the alignment, the individual alignment feature is compared with the average feature of a roadway (Fitzpatrick et al. 2000b).

Table 5.2 Details of data collected and alignment indices for individual curves

Section	Notation	R (m)	Δ (deg)	D (deg)	L_H (m)	PTL(m)	DTL(m)	G1 %	G2 %	A %	Number of accidents			Alignment indices	
											FA	GI	SI	R/AR	T/ATL
1	H1	219	40	7.8	153.0	110	0	-	-		1	2	2	1.2	1.7
	AH1	199	48	8.6	168.0	0	0	-	-		-	-	-	1.0	0.0
	H2	181	60	9.5	190.0	0	112	-	-		-	1	-	1.0	0.0
	CC1	162	61	10.6	173.0	112	94	4.3	-0.6	4.9	-	-	-	0.9	1.8
						94					-	-	-		1.5
2	AH2	130	55	13.2	126.0	60	59	-	-		-	-	-	0.9	0.6
	H3	176	56	9.8	174.0	59	230	-	-		-	5	1	1.2	0.6
	H4	115	63	14.9	127.0	230	300	-	-		-	-	2	0.8	2.3
	V1	-			395.0	300	100	5.0	-4.4	9.4	-	-	-		3.0
	H5	256	33	6.7	149.0	100	0	-	-		-	-	-	1.8	1.0
	AH3	58	63	29.6	64.0	0	0	-	-		-	2	-	0.4	0.0
	AH4	126	25	13.6	56.0	0	60	-	-		1	-	-	0.9	0.0
						60					-	-	-		0.6
3	H6	56	87	30.7	85.0	74	200	-	-		-	1	-	0.41	0.87
	CC2	129	49	13.3	111.0	200	120	4.7	-0.5	5.2	-	1	-	0.93	2.34
	H7	160	41	10.7	114.0	120	0	-	-		-	-	-	1.16	1.40
	AH5	184	17	9.3	55.0	0	0	-	-		-	-	-	1.33	0.00
	H8	145	41	11.9	103.0	0	100	-	-		-	1	-	1.05	0.00
	CC3	125	48	13.8	104.0	100	110	-2.7	-0.9	1.8	-	-	-	0.90	1.17
	H9	168	44	10.2	129.0	110	80	-	-		-	-	-	1.22	1.29
						80					-	-	-		0.94

Table 5.2 Details of data collected and alignment indices for individual curves (Continued…)

	Notation	R (m)	Δ (deg)	D (deg)	L_{H}(m)	PTL(m)	DTL(m)	G1 %	G2 %	A %	FA	GI	SI	R/AR	T/ATL
4	H10	368	16	4.7	100.0	360	250	-	-		1	1	-	1.1	1.5
	H11	300	14	5.7	75.0	250	115	-	-		-	-	2	0.9	1.0
						115					-	-	-		0.5
	H12	188	26	9.1	85.0	120	100				-	1	-	1.7	1.1
	H13	148	34	11.6	87.0	100	105	-	-		-	1	-	1.3	0.9
	CC4	65	54	26.4	61.0	105	75	2.1	-0.6	2.7	-	-	1	0.6	1.0
5	H14	150	42	11.5	111.0	75	160	-	-		-	-	1	1.3	0.7
	H15	105	58	16.4	107.0	160	100	-	-		-	1	-	0.9	1.5
	H16	70	53	24.6	65.0	100	56	-	-		-	-	-	0.6	0.9
	AH6	52	97	33.1	88.0	56	78	-	-		-	-	-	0.5	0.5
	H17	117	60	14.7	122.0	78	180	-	-		-	-	1	1.0	0.7
						180					-	-	-		1.7
	H18	83	99	20.7	143.0	120	0	-	-		-	-	-	0.7	2.2
	AH7	121	75	14.2	158.0	0	0	-	-		-	-	1	1.0	0.0
	AH8	116	57	14.8	115.0	0	0	-	-		-	-	-	1.0	0.0
	V2	-			100.0	60	120	2.5	-3.2	5.7	-	-	-		1.1
	CC5	165	59	10.4	170.0	120	15	6.1	-0.5	6.6	-	-	-	1.4	2.2
6	CC6	80	86	21.5	120.0	15	0	6.3	-0.6	6.9	-	-	-	0.7	0.3
	CC7	58	101	29.6	102.0	0	0	5.5	-0.5	6.0	-	-	-	0.5	0.0
	H19	102	65	16.9	116.0	0	230	-	-		-	-	-	0.8	0.0
	H20	256	40	6.7	180.0	230	0	-	-		-	1	-	2.1	4.2
	CC8	111	50	15.5	96.0	0	55	2.4	-0.5	2.9	-	-	-	0.9	0.0

Table 5.2 Details of data collected and alignment indices for individual curves (Continued…)

						55					-	-	-		1.0
	H21	223	23	7.8	90.0	0	100	-	-		-	-	-	1.0	0.0
7	V3	-			120.0	100	90	3.7	-3.2	6.9	-	1	-		1.6
						90					-	-	-		1.4
	H22	169	46	10.3	137.0	110	78	-	-		-	-	-	1.2	1.0
8	AH9	79	86	22.1	119.0	78	150	-	-		-	-	-	0.6	0.7
	H23	165	42	10.6	122.0	150	84	-	-		-	1	1	1.2	1.4
						84					-	-	-		0.8
	H24	185	67	9.4	215.0	150	110	-	-		-	1	-	1.0	1.9
	V4	-			150.0	110	34	3.8	-3.0	6.8	-	-	-		1.4
	AH10	76	47	23.0	62.0	34	0	-	-		-	-	-	0.4	0.4
9	CC9	268	30	6.5	142.0	0	92	3.6	-0.5	4.1	-	-	-	1.5	0.0
	H25	180	14	9.7	45.0	92	100	-	-		-	-	-	1.0	1.1
						100					-	-	-		1.2
	V5	-			200.0	90	120	3.5	-2.5	6.0	1	-	-		1.2
	H26	195	65	8.8	220.0	120	0	-	-		1	1	-	0.9	1.5
	H27	155	68	11.1	185.0	0	140	-	-		-	1	-	0.8	0.0
	AH11	225	56	7.6	219.0	140	0	-	-		-	-	-	1.1	1.8
10	AH12	315	47	5.5	257.0	0	150	-	-		-	1	-	1.5	0.0
	H28	130	81	13.2	184.0	150	0	-	-		-	-	3	0.6	1.9
	AH13	86	62	20.0	93.0	0	110	-	-		-	-	1	0.4	0.0
	V6	-			170.0	110	40	3.6	-2.5	6.1	-	-	-		1.4
	AH14	314	35	5.5	193.0	40	65	-	-		-	-	-	1.5	0.5
	H29	228	56	7.5	224.0	65	140	-	-		-	-	-	1.1	0.8
						140									1.8

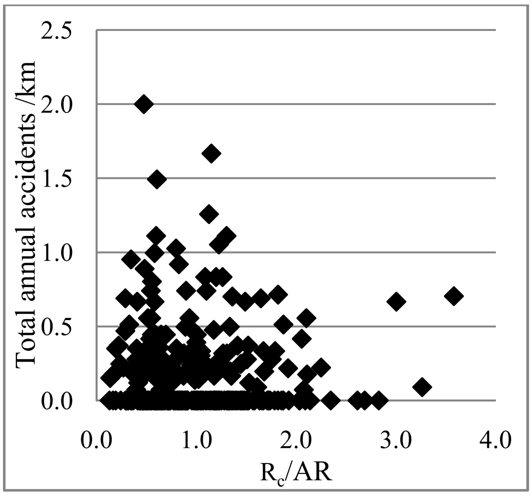
The alignment indices such as R_c/AR and TL/ATL are only two indices can be useful for this purpose because all other alignment indices are length-based, which will provide more information related to the consistency of a section of a roadway than as an individual measure.

On rural highways more than 30% of the total fatality accidents take place on curved sections than on straight segments (Gibreel et al.1999). Thus, curved section and the corresponding transition sections represent the most critical locations, while considering measures for improvement of highway safety. Therefore, the comparison between radius of curve with average radius of curves in the stretch appeared to relate well to both speed and accident values. R_c/AR determines which curve appears to be in conflict with the general character of the horizontal curves of the alignment. Large variation in R_c/AR indicates large change in speed required at curves, which may lead to inconsistent locations.

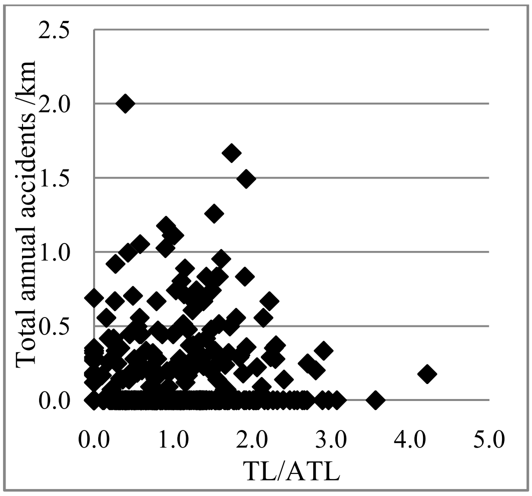
The length of tangent determines the speed motorists will reach on that tangent. If a tangent is long enough, then motorists will drive at their desired speed. If the motorists are driving at a high speed on the tangent and a large reduction in speed is required at the following curve or short tangent in a stretch, they may not be able to decrease their speed as needed, which may lead to inconsistent situations. Therefore, a comparison of individual tangent length to the average tangent length appears to be very valuable in rating the design consistency of the roadway. It shows the amount of variation for each individual tangent and can possibly be used in determining locations where motorists may expect the tangents to be longer than they are.

Therefore, to find the relationship between the alignment indices R_c/AR and TL/ATL with safety, these indices are analysed with annual accidents/km. Figs. 5.1 to 5.4 provides the information about variables and their justification for use in consistency evaluation of individual features.

The Fig.5.1(a) provides a good visual indication of where differences in radii exist among the individual curves. Fig.5.1(b) shows the amount of variation for each individual tangent and can possibly be used in determining locations where motorists may expect the tangents to be longer than they are. It can be seen from Fig.5.1 (a) and Fig.5.1(b), that though the data are clustered, it is clear that the total number of annual accidents/km is high when the ratio of R_c/AR is between 0.5 and 1.4, and TL/ATL ratio is between 1.0 and 2.0.



(a)

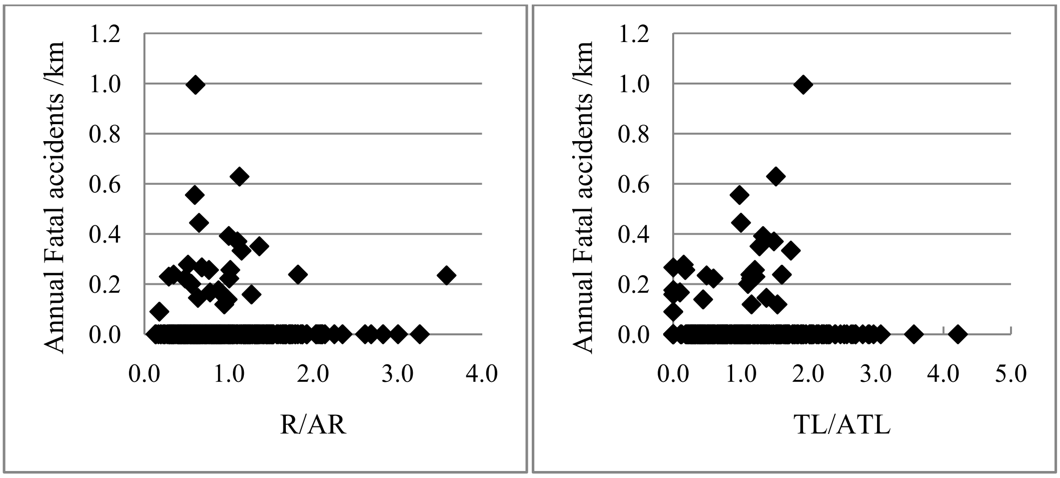


(b)

Fig.5.1 Variation of Total annual accidents/ km with alignment indices

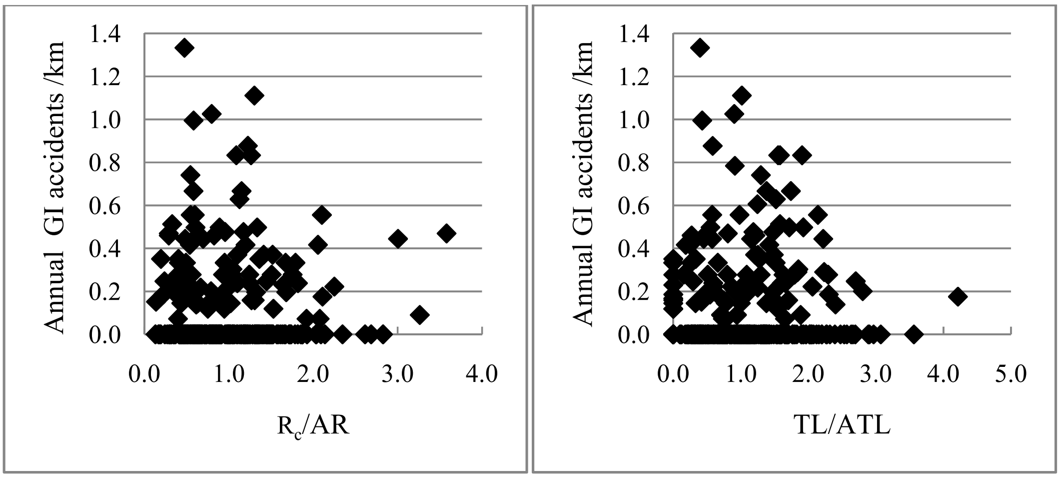
Fig.5.2 shows the variation of annual fatal accidents/ km with alignment indices. From the Figures 5.2 (a) and 5.2 (b), it is clear that annual fatal accidents/ km are less, when R_c/AR and TL/ATL attain value one.

The variation of annual grievous injury accidents/ km with alignment indices is given in Figures 5.3 (a) and 5.3 (b) also shows similar trend as in Figures 5.2 (a) and 5.2 (b) respectively.



(a) (b)

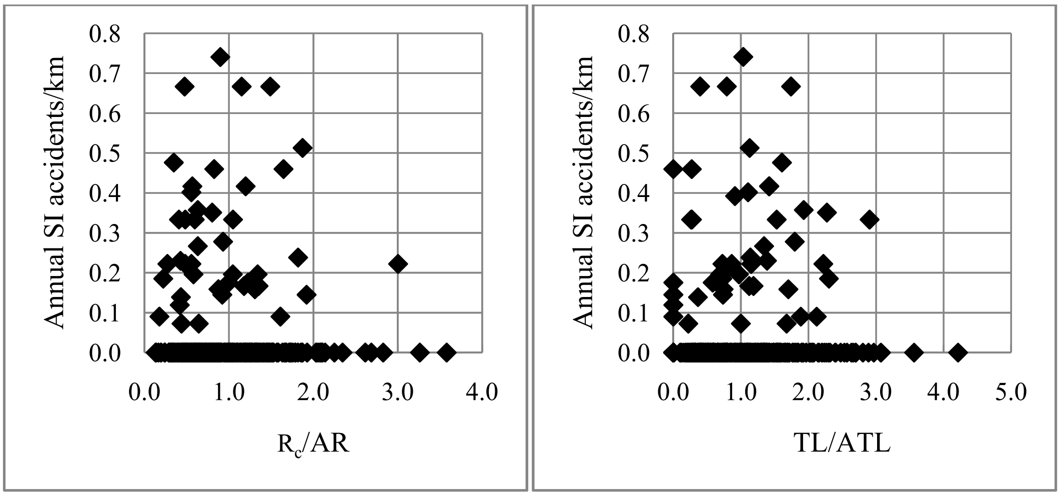
Fig. 5.2 Variation of Annual Fatal accidents/ km with alignment indices



(a) (b)

Fig. 5.3 Variation of Annual Grievous Injury (GI) accidents/ km with alignment indices

It can be observed from Figs.5.4 (a) and Fig.5. 4 (b) that the data are clustered and do not show clear thresholds to explain the relationship between annual simple injury accidents/km with alignment indices considered for individual elements.



(a) (b)

Fig. 5.4 Variation of Annual Simple Injury (SI) accidents/ km with alignment indices

This indicates that geometric variables may not directly cause simple injury accidents. Therefore, the simple injury accidents may be due to variation in driver or vehicular characteristics.

5.10 RELATIONSHIP BETWEEN SAFETY AND ALIGNMENT INDICES

To recommend alignment indices as a design consistency measure, it would be valuable to relate alignment indices with safety. Therefore, initially it is necessary to determine whether any of the alignment indices was able to individually predict annual accidents/km of intermediate lane rural highways. The alignment indices selected for evaluation as potential design consistency measures include:

- Horizontal alignment indices
    - Curvature change rate (CCR)
    - .Degree of curvature (DC)
    - Ratio of curve length to length of a highway section (CL/RL).
    - Average radius (AR) of a highway section.
    - Average tangent length (ATL) of a highway section

- Ratio of maximum radius of curvature to minimum radius of curvature of a highway section (RR).
- Minimum value of the ratio of an individual curve radius to the average radius of the highway section, $(R_c/AR)_{min}$, which is an inverse of AR/R_{min} of the highway section.
- Maximum value of the ratio of an individual tangent length to the average tangent length,$(TL/ATL)_{max}$, of a highway section.

❖ Vertical alignment indices
- Vertical curvature change rate (VCCR) of a highway section.
- Average rate of vertical curvature (AVC) of a highway section.

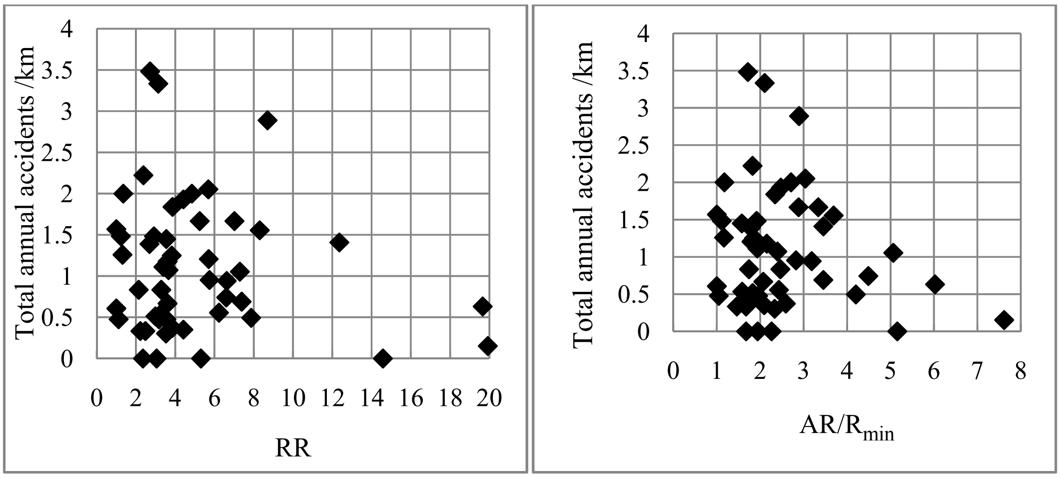
❖ Composite alignment indices
- Combination curvature change rate (COMBO)) of a highway section.

The length-based horizontal alignment indices such as CCR, DC, AR, ATL, and CL/RL, vertical alignment indices such as VCCR, AVC and composite alignment index, and COMBO provide more information related to the consistency of a section of roadway than as an individual measure (Fitzpatrick et al. 2000a). The length-based horizontal alignment indices are tabulated in Table 5.3. In this study horizontal alignment index RR and AR/R_{min} are used to determine the design consistency of the alignment, and other length-based horizontal alignment indices such as CCR, DC, AR, ATL, CL/RL,VCCR, AVC and COMBOare used to interpret the variation in the general character of the alignment (may not be expected to provide information on individual features of the roadway in attempting to determine those features that may be inconsistent).

The length of the alignment characteristics that affect motorists, and thereby accidents was selected between 0.5 km and 4.6 km. The lower value of 0.5 km was selected because the number of geometric features encountered by motorists within 0.5 km was considered to be too small for motorists to create any expectations of the upcoming roadway. The higher value of 4.6 km was selected, as it was assumed that the indices would not be affected much by changes in the upstream alignment characteristics. In addition to this, in this study, the maximum length of a highway section considered is of 4.6 km. Therefore, the stretches whose length is less than 0.5 km were removed from data set. This process reduced the number of highway sections/stretches to 51. Using these data, the graphical analysis was performed to find out the relationship between road safety and alignment indices.

5.10.1 Graphical Analysis

The ability of individual alignment indices to predict the annual accidents/km was first examined graphically. For all the calculated eleven alignment indices for the highway sections/stretches, graphs of the observed annual accidents/km against the alignment indices were developed. The graphs, shown in Figs. 5.5 to 5.10 provide a visual indication of any relationships that may exist between the safety and the alignment indices.



(a) (b)

Fig. 5.5 Variation of Total annual accidents/ km with alignment indices RR and AR/R_{min}.

The range of the radii along a roadway can be determined by computing the ratio of the maximum radius to minimum radius (RR). This ratio can represent the consistency of the design in terms of the use of similar horizontal radii along the road.

Table 5.3 Alignment Indices for the selected stretches

Section	Length (km)	AR (m)	ATL (m)	$(R_c/AR)_{min}$	$(TL/ATL)_{max}$	RR	CCR deg/km	DC deg/km	CL/RL	AR/Rmin	AVC m/%	VCCR deg/km	COMBO deg/km	TA
1	1	190.3	63.2	0.9	1.8	1.4	209.0	36.6	0.7	1.2	35.3	2.8	64.0	6
2	1.9	143.5	101.1	0.4	3.0	4.4	155.8	46.3	0.4	2.5	42.0	2.8	158.6	11
3	1.2	138.1	85.5	0.4	0.9	3.3	272.1	83.3	0.6	2.5	39.6	0.9	273.0	3
4	0.9	334.0	241.7	0.9	1.5	1.2	33.2	11.6	0.2	1.1	0.0	0.0	33.2	4
5	1.7	111.9	108.2	0.5	1.7	3.6	249.4	86.7	0.4	2.2	22.6	0.9	250.3	6
7	0.4	223.0	63.3	1.0	1.6	1.0	55.0	19.3	0.2		17.4	9.9	67.7	1
6	1.9	121.3	54.5	0.5	4.2	4.4	332.1	79.1	0.6	2.1	20.4	8.5	340.6	2
8	0.8	137.7	105.5	0.6	1.4	2.1	218.9	52.9	0.5	1.7	0.0	0.0	218.9	2
9	1.1	177.3	81.0	0.4	1.9	3.5	143.6	43.5	0.4	2.3	28.3	5.7	149.3	1
10	2.8	206.0	77.7	0.4	1.9	3.7	167.9	28.3	0.6	2.4	30.6	1.2	169.1	9
13	1.4	349.0	223.8	0.3	1.6	5.2	92.0	16.3	0.4	2.9	0.0	-	92.0	7
14	1.8	202.0	152.0	0.2	2.3	6.6	94.1	28.1	0.2	4.5	26.4	6.6	100.8	4
15	3.7	265.2	127.3	0.2	2.1	19.7	129.9	31.3	0.3	6.0	53.9	5.5	135.3	7
16	2.1	209.6	74.0	0.4	2.1	5.7	171.8	41.3	0.5	2.8	67.2	5.5	177.3	6
17	1.25	171.9	104.2	0.6	2.5	3.2	138.4	40.6	0.4	1.6	52.5	6.2	144.6	2
18	1.66	158.3	94.1	0.6	2.8	5.7	222.4	70.1	0.5	1.8	42.6	5.2	227.6	6
19	2.3	138.8	76.1	0.6	2.2	3.5	171.6	57.3	0.4	1.6	157.5	7.2	178.8	10
20	2	175.4	84.2	0.7	3.6	2.2	135.4	37.8	0.4	1.5	31.3	4.6	140.0	2
21	1	120.0	145.6	0.5	2.9	3.6	156.0	71.2	0.3	2.1	0.0	4.6	160.5	2
22	2.06	138.2	74.8	0.2	3.1	14.6	209.3	110.2	0.3	5.2	16.3	6.3	215.6	0

Table 5.3 Alignment Indices for the selected stretches (Continued…)														
Section	Length (km)	AR (m)	ATL (m)	$(R_c / AR)_{min}$	$(TL/ ATL)_{max}$	RR	CCR deg/km	DC deg/km	CL/ RL	AR/ Rmin	AVC m/%	VCCR deg/km	COMBO deg/km	TA
23	1.1	175.9	134.8	0.4	1.4	5.3	122.1	62.4	0.3	2.3	0.0	0.0	122.1	0
24	2.2	319.2	107.8	0.1	2.4	19.9	122.6	48.4	0.4	7.6	19.1	3.0	125.6	1
25	1	217.8	94.9	0.5	2.6	3.0	127.1	45.8	0.4	1.9	0.0	0.0	127.1	0
26	1.45	197.0	91.6	0.4	1.5	3.9	171.6	50.9	0.5	2.3	0.0	0.0	171.6	8
27	1.8	263.1	75.5	0.4	2.9	3.9	144.9	47.1	0.5	2.6	0.0	0.0	144.9	2
28	0.4	107.9	67.0	0.9	1.6	1.2	267.7	80.3	0.5		0.0	0.0	267.7	3
29	0.7	144.9	81.0	0.5	1.5	3.2	193.3	85.9	0.4	1.8	0.0	-	193.3	1
30	0.65	136.4	110.8	0.3	1.6	5.7	233.2	93.6	0.3	3.0	0.0	-	233.2	4
31	2.4	146.0	77.0	0.4	2.5	6.2	207.2	87.7	0.4	2.4	4.6	1.4	208.5	4
32	0.4	279.1	140.0	1.0	1.2	1.0	83.3	20.5	0.4		0.0	0.0	61.6	0
33	1.4	211.2	84.4	0.5	2.0	3.6	109.3	42.1	0.3	2.0	10.3	2.4	111.7	2
34	1.45	235.1	152.2	0.3	2.7	7.4	111.7	41.0	0.3	3.5	9.2	2.6	114.3	3
35	0.55	81.6	88.3	1.0	1.2	1.0	61.8	38.3	0.1	1.0	31.6	7.9	65.3	1
36	0.5	209.6	100.7	0.6	2.0	2.4	126.1	39.2	0.4	1.7	0.0	-	126.1	0
37	4.6	178.1	136.0	0.3	2.3	6.6	144.5	47.7	0.4	3.2	5.9	1.0	145.5	13
38	0.6	168.0	98.5	0.5	2.1	2.4	131.0	59.0	0.3	1.8	0.0	0.0	131.0	4
39	0.7	66.9	137.3	1.0	1.2	1.1	186.0	110.4	0.2	1.0	0.0	-	186.0	1
40	1.5	158.1	121.6	0.3	2.2	8.7	155.4	79.6	0.4	2.9	16.3	0.3	155.7	13
41	0.5	68.2	50.6	0.5	2.0	3.1	522.0	261.3	0.5	2.1	0.0	0.0	522.0	5
42	1.2	265.3	125.2	0.6	1.8	2.7	75.9	19.0	0.3	1.8	36.3	2.4	78.3	5

Table 5.3 Alignment Indices for the selected stretches (Continued…)

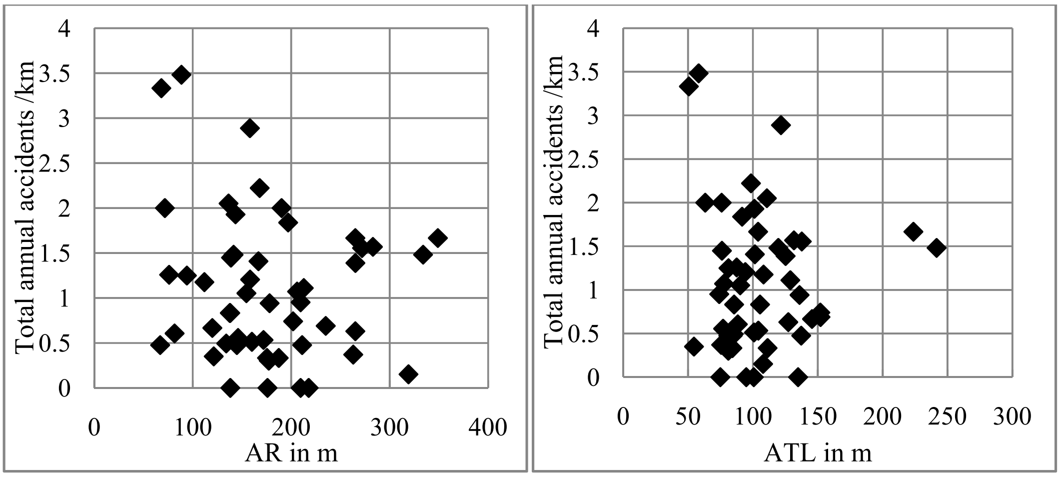
Section	Length (km)	AR (m)	ATL (m)	$(R_c / AR)_{min}$	$(TL/ATL)_{max}$	RR	CCR deg/km	DC deg/km	CL/ RL	AR/ Rmin	AVC m/%	VCCR deg/km	COMBO deg/km	TA
43	0.85	283.0	131.7	1.0	1.3	1.0	34.5	7.2	0.2	1.0	36.5	5.2	45.7	4
44	0.4	80.4	90.7	0.7	1.9	1.7	242.1	114.9	0.3		0.0	-	242.1	1
45	0.53	76.0	87.5	0.9	1.8	1.3	257.5	129.6	0.3	1.2	0.0	-	257.5	2
46	0.8	94.1	81.2	0.5	2.2	3.8	287.2	149.8	0.4	1.9	0.0	0.0	287.2	3
47	1	71.7	75.9	0.4	2.6	4.8	414.3	254.5	0.4	2.7	0.0	0.0	414.3	6
48	2	187.4	111.3	0.6	2.7	2.5	127.8	36.0	0.4	1.7	28.0	2.1	130.0	2
49	0.9	141.5	119.6	0.5	1.3	2.9	160.2	69.1	0.3	1.9	0.0	-	160.2	4
50	1.42	166.8	101.5	0.3	2.0	12.4	146.9	88.6	0.3	3.5	29.4	3.2	150.1	6
51	1.5	272.0	137.7	0.3	2.1	8.3	98.7	40.0	0.4	3.7	0.0	-	98.7	7
52	0.67	88.5	58.2	0.6	1.9	2.7	365.2	170.1	0.5	1.7	0.0	0.0	365.2	7
53	1.3	160.1	100.9	0.5	1.7	3.0	175.9	55.7	0.5	1.8	0.0	0.0	175.9	2
54	1.35	134.1	85.1	0.2	2.7	7.9	230.8	109.8	0.4	4.2	0.0	0.0	230.8	2
55	0.95	154.6	90.2	0.2	1.8	7.3	149.3	87.7	0.3	5.1	28.6	3.0	152.2	3
56	1.2	212.8	128.8	0.5	1.9	3.4	105.1	40.7	0.4	1.9	0.0	0.0	105.1	4
57	0.6	265.2	104.0	0.3	1.9	7.0	90.9	59.3	0.3	3.3	0.0	0.0	90.9	3

Polus (1980) found that as this value approaches one (i.e. as the consistency of the chosen design radii increases), a reduced accident rate may be expected. From the Fig.5.5 (a) it is clear that total number of annual accidents is less when RR is one, which agrees with Polus (1980) study results. This states that if the ratio of the individual radius to the maximum radius is similar for all radii, it indicates that there is consistency among the curves along the roadway. It is also observed from the Fig.5.5 (a) that the curve in the section of a highway is more inconsistent when RR value is ranges between 2 to 4. A further increase of RR shows reduced number of accidents. As the maximum radius is much different from all other radii, the curve with the maximum radius may cause inconsistent location, but may not lead to accidents. Therefore, even though RR shows a better representation of the variation of radii along a highway, it is better to use some other indices as radii measure to strengthen this measure..

The ratio of AR/R_{min} also can represent the consistency of the design in terms of the use of similar horizontal radii along the road. This ratio can be used to identify the inconsistent curve, which deviates from average radius of curves along a section of the highway. It is observed from the Fig.5.5 (b) that a curve in a section of highway is more inconsistent when AR/Rmin value ranges from 1.5 to 2.5.

The average radius expresses what motorists typically encounter on curved sections of the road. A large Average Radius (AR) would indicate curves that are typically not very sharp. Therefore, it is expected that higher speeds would exist for these values compared to smaller average radius values. The Average Tangent Length (ATL) indicates the length of tangent that is typically available to motorists between curved sections of the roadway. A large value for this index would indicate that the road has tangent sections that are typically long; therefore, speed of vehicles would be expected to be higher than for roads with a smaller value.

From the Figs.5.6 (a) and (b) it is observed that as AR or ATL increases, a decreasing trend of total annual accidents/km is noticed. When AR or ATL increases means, it means it provides a good platform for the driver to drive, thereby reducing the mental workload, which may be the reason for reduced accidents.

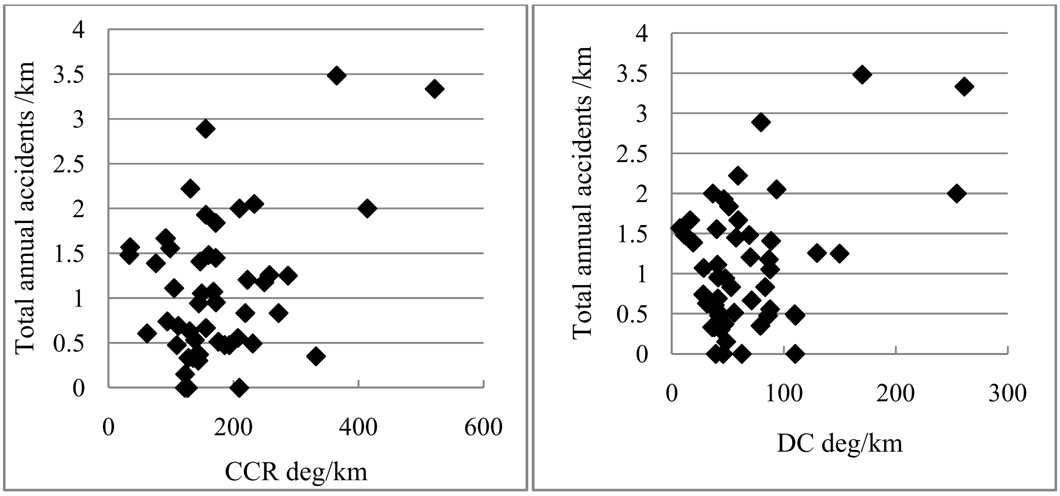


(a) (b)

Fig 5.6 Variation of Total annual accidents/ km with alignment indices AR and ATL

In Fig.5.6 (a) it is observed that a large number of accidents have occurred, when AR is between the ranges of 125 m-225 m. In Fig.5.6 (b) it is the observed that a more number of accidents have occurred, when ATL is between the ranges of 75 m-110 m The value of ATL could indicate the possibility of safety problems on a roadway section if tangents were so long as to become monotonous to the motorist or if a sharp curve were located at the end of the long tangent. On the other hand, a short average tangent length could imply that the roadway consists of a series of short curves and tangents, while a longer average tangent length may indicate a more generous design. Thus, the interpretation of average tangent length and average radius appears ambiguous.

A large value for the indices CCR and DC indicates that the road either contains a large number of curves or there are long or sharp curves in that section. It was expected that an increase in the value of these indices would increase the workload of a driver resulting in decrease of reaction towards the problem encountered during the journey, thereby increasing the number of accidents. From the Figs.5.7 (a) and (b), it is clear that the expectation is true as it also shows a better relationship with annual accidents/km compared to all other indices considered.

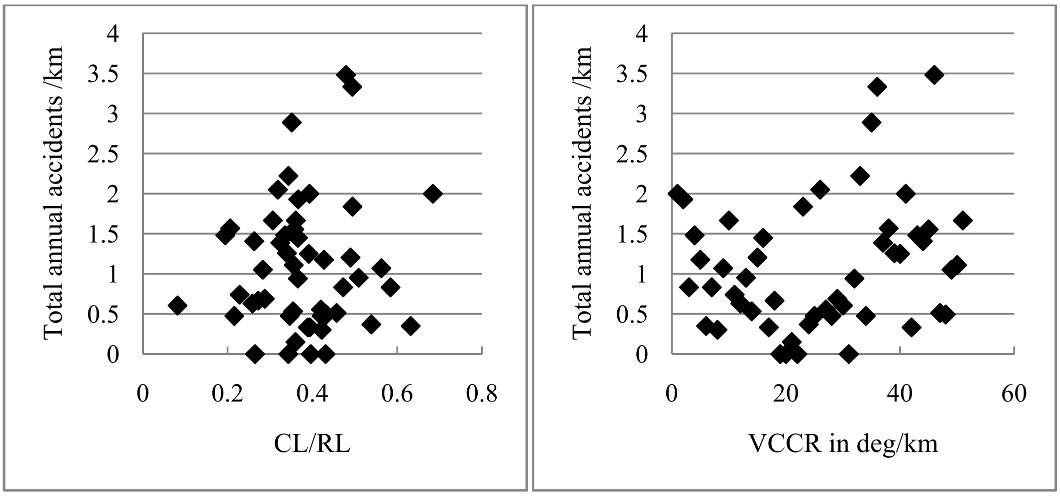


(a) (b)

Fig 5.7 Variation of Total accidents/ km with alignment indices CCR and DC

The ratio of curve length to total roadway length provides a good indication of the character of the alignment. This alignment index provides information on the proportion of the roadway that is on curved sections. As the road includes more curves or longer curves, this proportion increases. From the Fig.5.8 (a) it is confirmed that when the road includes more curves or longer curves, it decreases the reaction time of a driver resulting in increase of total annual accidents/km.

The algebraic difference in grade per length measures the angular change in the vertical direction of a road. A large Vertical CCR (VCCR) value would indicate the existence of many changes in the vertical direction of a roadway or the presence of steep grades. From the Fig.5.8 (b), it is not possible to draw any conclusion towards the effect of VCCR on total annual accidents/km.



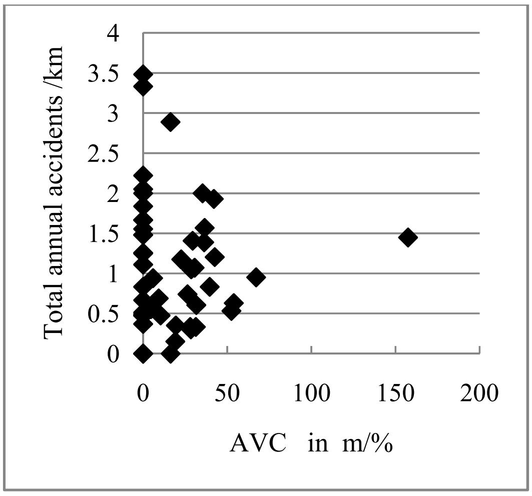
(a) (b)

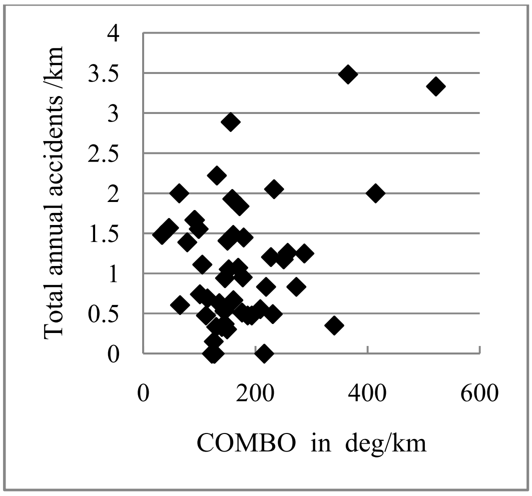
Fig 5.8 Variation of Total accidents/ km with alignment indices CL/RL and VCCR

The average rate of vertical curvature was used as an alignment index because it provides a measure of the sharpness of the vertical curves of a road. It was thought that this index provides a good indication of the vertical character of the alignment. A large value for this index would indicate that motorists are provided with more sight distance when travelling on the vertical curves of the roadway. Therefore, the effects of a steep grade, if present, may be reduced, and increased sight distance results in decrease of total annual accidents/km. This trend is observed in Fig.5.9 (a), but the relationship of AVC with total annual accidents/km is poor.

When there is increase in the value of the combination CCR (COMBO), it indicates more angular changes in either the horizontal and/or vertical direction of the roadway. From Fig.5.9 (b), it is observed that the total annual accidents/km would tend to increase as the value of the combination CCR increases. The more angular changes of the roadway may confuse the expectation of a driver leading to hazardous situations, which may be the reason for higher total annual accidents/km.

The vertical alignment indices such as VCCR and AVC, and composite alignment index, COMBO, provide information related to the consistency of a section of roadway but it is difficult to interpret the consistency criteria. Table 5.4 presents the results obtained from the graphical analysis, based on alignment indices calculated for intermediate lane rural highways.





(a) (b)

Fig 5.9 Variation of Total accidents/ km with alignment indices AVC and COMBO

Table 5.4 Consistency results obtained based on alignment indices

Feature	Consistency Measure	Criterion	Evaluation
Individual	R_c/AR	Between 0.5 to 1.4	More inconsistent
	TL/ATL	Between 1.0 to 2.0	
Stretch	R_{max}/R_{min} (RR)	Between 2 to 4.	
	AR/R_{min}	Between 1.5 to 2.5	

The result presented in the Table 5.4 does not match with results obtained by Polus and Dagan (1987), Anderson et al. (1999), and Fitzpatrick et al. (2000a). Therefore ,it can be concluded that alignment indices hold good for design practice in the United States, England, and Germany. However, this method does not show similar results in consistency evaluation of intermediate lane rural highways, which may be due to many variations among the geometrics of highways that exist in India.

5.11. SUMMARY

Several alignment indices were identified as possible quantitative measures in rating the design consistency of intermediate lane rural highways. An analysis of safety was done to test the relationship with the alignment indices identified as preliminary design consistency measures. R_c/AR and TL/ATL are found to be a better measure to identify the design consistency of individual features. RR and AR/R_{min} are observed to be a better measures to identify dissimilarity exists in stretch of intermediate lane rural highways.

CHAPTER 6

SUMMARY, CONCLUSIONS AND FUTURE SCOPE

6.1 SUMMARY

India is witnessing extensive modernization and improvement of its road network, particularly with regard to condition of pavement and road safety. From a planning and design perspective, the road geometry factors are of primary interest. Better design consistency guidelines are needed so that designers can review the effects of successive roadway features on drivers and eliminate (or minimise) the presence of successive features that require large speed adjustments by drivers.

On literature study it is found that more accidents occur at curves and speed at curves mainly depends on geometry of the highways. Out of several consistency methodologies, speed-based consistency is the common method of consistency evaluation of highways. Alignment indices are the non-speed based method of consistency evaluation and are used as a supplementary method with speed-based consistency evaluation.

In Dakshina Kannada district of Karnataka state, India, more than 50% of state highways are of intermediate lane and accident situation in the district is relatively more serious owing to the rapid growth of vehicle population and inadequacy of road-related characteristics. Geometric design evaluation can be used to pinpoint locations on highways where accidents could considerably be higher.

The objective of this study is to evaluate design consistency of both horizontal alignment and vertical alignment and development of evaluation of consistency criteria using speed-based method and alignment indices. The basic data required for consistency evaluation of highway are geometric, speed, and accident data. In this study the geometrics were measured either at the field or extracted from CAD drawings prepared using surveying details. Spot speed study was conducted to collect speed details of passenger cars during off-peak periods and dry weather conditions at study points. Accident data was collected from various police stations and accident spots were identified and marked on the alignment drawings.

Operating speed models at study points of horizontal curve and speed differential models between tangent and curve are developed using geometrics of highways. To predict the most significant models horizontal curves are classified based on shoulder width available at midpoint of curve.

According to this study, the radius of a horizontal curve is an important factor influencing a driver’s decision. In addition to this, sight distance available before the study point also have the influence on operating speed at study points Operating speed models are also developed for crest vertical curves. Approaching gradient and preceding tangent length are reflected as explanatory variables to predict operating speed at limit and summit point of crest vertical curves. The operating speed at tangent point of both horizontal and crest vertical curves are affected by Preceding tangent length.

Design consistency evaluation criteria were developed for single and successive elements of horizontal curves and vertical curves. Alignment indices are the quantitative measures used to quantify the variations of existing curves in a section of intermediate lane rural highways. Also, an attempt is made to find out the relationship between road safety and alignment indices of intermediate lane rural highways.

6.2 CONCLUSIONS

From the consistency evaluation of intermediate lane rural highways, following conclusions are drawn:

1) 65.3 % of the total accidents on intermediate lane rural highways are occurred on curved sections. Hence it can be concluded that the curved section and the corresponding

transition sections represent the most critical locations while considering the measures for improvement of highway safety.

2) From the observed trend of operating speed at study points of the horizontal curve it can be concluded that the drivers tend to decrease their speed while approaching the midpoint of the horizontal curve, where the sight distance is limited, and then increase as vehicle approaches the end of curve.

3) From the observed trend of operating speed at study points of vertical curve, it can be concluded that the drivers tend to decrease their speed while approaching the limit point of the vertical curve, where the sight distance is limited due to the gradient of road and as the vehicle approaches the summit point of the vertical curve, the sight distance starts to increase and thus the drivers tend to increase their speed.

4) From the analysis of accidents, it is concluded that the radius of less than 250 m is very dangerous because the number of accidents and severity of accidents are high. As the radius of the horizontal curve decreases, the available sight distance also decreases and speed reduction between the tangent and the curve increases, thus resulting in more accidents.

5) The geometric variables influencing operating speeds at various speed observation points are found to be as follows:

a) Horizontal curves,

- The operating speed of the curve and the speed differential between the tangent and the curve are mainly affected by carriageway and shoulder width available at the middle of curve.
- The operating speed at tangent point (V_{TM}) is influenced by inverse of square root of preceding tangent length ($1/\sqrt{PTL}$), and the available sight distance at start of tangent (SD_{TS}).
- The operating speed at midpoint of Class A curves is influenced by the logarithmic of reciprocal of radius ($\ln(1/R)$), and the available sight distance at start of curve (SD_S).

- The operating speed at midpoint of Class B curves is influenced by square root of radius ($\sqrt{R}$), and the available sight distance at start of curve (SD_S).
- The operating speed at midpoint of Class C curves is influenced by the inverse of radius (1/R), and the available sight distance at start point of curve (SD_S).
- The radius of the curve has a great effect on the speed differential between tangent and curve, for the curve radii less than approximately 250 m, but this effect tends to vanish after the 400 m radius in intermediate lane rural highways.
- The ratio of preceding tangent length (PTL) and the radius of the curve (R) are having significant effect on the speed differential between tangent and midpoint of Class A curves.
- The ratio of preceding tangent length (PTL) and square root of radius of the curve ($\sqrt{R}$) are having significant effect on the speed differential between tangent and midpoint of Class B curves.

b) Vertical curves,

- The operating speed at tangent point (V_{TM}) of vertical curve is influenced by the approach tangent length.
- The operating speed at limit point (V_{LP}) increases as the approach tangent length (PTL) increases and decreases as the approach gradient (G1) increases.
- The operating speed at summit point (V_{SP}) of vertical curve is influenced by the length of preceding tangent (PTL) and the approach gradient (G1).

6) The consistency criteria developed are as follows:

a) Horizontal curves

- Based on the design consistency criteria developed for a single element of horizontal curve, it can be concluded that a good design consistency can be achieved when the difference between the operating speed at middle of curve and the design speed does not exceed 10 km/h.

- Based on the design consistency criteria developed for the successive elements of horizontal curve, it can be concluded that a good design consistency can be achieved when the difference between the operating speeds on the tangent and at middle of the following curve does not exceed 15 km/h.

b) Vertical curves

- Based on the design consistency criteria developed for a single element of crest vertical curve, it can be concluded that a good design consistency can be achieved when the difference between the operating speed at limit point and the design speed does not exceed 36 km/h.
- The design of crest vertical curves in intermediate lane rural highways is said to be good, satisfactory, and poor, when the differences between operating speeds at tangent point and limit point are (V_{TM}-V_{LP}) is <10 km/h, 10-20 km/h and > 20 km/h respectively.

6) Consistency evaluation by alignment indices:

- The alignment indices such as R/AR, TL/ATL are selected as potential design consistency measures for consistency evaluation of individual alignment. It can be concluded that individual alignment is said to be inconsistent, when the ratio of R/AR is between 0.5 and 1.4, and TL/ATL ratio is between 1 and 2.
- Ratios of R_{max}/R_{min} (RR) and AR/Rmin are used to identify the variation of the horizontal feature in a section. It is concluded that the curve in an alignment is said to be inconsistent when RR is more than 2 and less than 4, and that a curve in a highway is more inconsistent when value AR/Rmin ranges from 1.5 to 2.5.
- Length-based horizontal alignment indices such as AR, ATL, and CL/RL are used to interpret the design consistency of the alignment.

- The vertical alignment indices such as Vertical CCR (VCCR), Average Rate of Vertical Curvature (AVC), and composite alignment index (COMBO) provide information related to the consistency of a section of roadway but is difficult to interpret the consistency criteria.
- Alignment indices hold good for design practice in the United States, England, and Germany. However, this method does not show good results in consistency evaluation of intermediate lane rural highway, may be due to much variations between the geometrics of highway that exist in India**.**

6.3 FUTURE SCOPE

1. Additional insight into the influence of speeds on curve section and also with grades is needed.
2. Effect of curve before the tangent needs to be studied to predict the speed on tangent.
3. Further research should be conducted in all aspects of the current research on roadway types other than intermediate lane rural highways.
4. Further research should be conducted in estimating operating speeds and 85th speed differential of trucks and other types of vehicles for different horizontal and vertical curves.

REFERENCES

Abidin, Z.N. and Adnan, M.A. (2008). "Design consistency models on combined horizontal and vertical alignments: case study." *EASTS International Symposium on Sustainable Transportation incorporating Malaysian Universities Transport Research Forum Conference* (MUTRFC08), University Technology Malaysia, pp. 12-13.

Al-Masaeid, H. R., Hamed, M., Aboul-Ela, M. and Ghannan, A. G. (1995). "Consistency of horizontal alignment for different vehicle classes." *Transportation Research Record* 1500, Transportation Research Board, National Research Council, Washington, D.C., pp. 178–183.

Anderson, I. B. and Krammes, R. A. (2000). "Speed reduction as a surrogate for accident experience at horizontal curves on rural two-lane highways." *Transportation Research Record* 1701, Transportation Research Board, Washington, D.C., pp. 86-94.

Anderson, I. B., Karin, M. B., Douglas, W. H. and Fitzpatrick, K. (1999). "Relationship to safety of geometric design consistency measure for rural two lane highways." *Transportation Research Record* 1658, Paper No-0921,Transportation Research Board, Washington, D.C., pp. 43-51.

Annual Report 2010-11, http://morth.nic.in/writereaddata/mainlinkFile/File414.pdf.

Babkov, V. F. (1975). "Road conditions and traffic safety." *Mir publishers*, Moscow.

Bonneson, J. A. (1999). "Side friction and speed as controls for horizontal curve design." *ASCE Journal of Transportation Engineering*, Vol. 125, No. 6, November/December, Paper No. 20291, pp. 473–480.

Cafiso,S., Alessandro, D. G. and Cava, G. L. (2005). "Actual driving data analysis for design consistency evaluation." *Transportation Research Record*, No. 1912, Transportation Research Board, Washington, D.C., pp. 19–30.

Caroline, M. H., Polus, A. and Farah, H. (2008). "Further evaluation of the relationship between enhanced consistency model and safety of two-lane rural roads in

Israel and Germany.” *EJTIR, 8(4),* ISSN: 1567-7141, www.ejtir.tbm.tudelft.nl, pp. 320-332.

Collins, J., Fitzpatrick, K., Karin, M. B. and Douglas, W. H. (1999). “Speed variability on rural two lane highways.” *Transportation Research Record* 1658, Paper No .0457, pp. 60-69.

Collins, K. M. and Krammes, R. A. (1995). “Preliminary validation of a speed profile model for design consistency evaluation.” *Transportation Research Record* 1523, Transportation Research Board, Washington, D.C., pp 11-21.

CRRI (1982). “Road user cost study in India.” Final Report,*Central Road Research Institute,* New Delhi.

Duong, D., Michael, M., Frank, S. and Bruce, H. (2010).“Assessing speed consistency of horizontal geometric design using a microscopic simulation approach.” *Proceedings of the 20th Canadian Multidisciplinary Road Safety Conference,* Niagara Falls, June 6-9.

Easa, S. M. (2003). “Improved speed-profile model for two lane rural highways.” *Canadian Journal of Civil Engineering*, 30(6), pp. 1055-1065.

Fambro, D. B., Urbanik, T., Hinshaw, W. M., Hanks, J. W., Ross, M. S., Tan, C. H. S.and Pretorius, C. J. (1999).“Stopping sight distance considerations at crest vertical curves on rural two-lane highways.” Report *FHWARD- 99-XX*. FHWA, U.S. Department of Transportation, Washington, D.C.

Figueroa, M. A. and Tarko, P. A. (2007). “Speed changes in the vicinity of horizontal curves on two lane rural roads.” *Journal of Transportation Engineering*, Vol. 133, No. 4, pp. 215-222.

Fitzpatrick, K., Carlson, P., Brewer, M. A., Wooldridge, M.D. and Shaw-Pin, M. (2003). “Design speed, operating speed and posted speed practices.” *NCHRP 504. Transportation Research Board,* Washington. D.C.

Fitzpatrick, K. and Collins, J. M. (2000). “Speed-profile model for two-lane rural highways.” *Transportation Research Record* 1737, Paper No. 145, National Research

Council, Washington D.C., pp. 42-49.

Fitzpatrick, K., Elefteriadou, L., Douglas, W. H., Collins, J. M., McFadden, J., Anderson, I. B., Krammes, R. A., Irizarry, N., Parma, K. D., Passetti, K. and. Bauer, K. M. (2000a). “Speed Prediction for Two-Lane Rural Highways.” Draft Final Report, *FHWA-RD-99-171*.

Fitzpatrick K., Green, P., Wooldridge, M. D., Tsimhoni, O., Collins, J. M., Bauer, K. M., Parma, K. D., Koppa, R., Harwood, D. W., Anderson, I. B., Krammes, R. A. and Poggioli. B. (2000b). “Alternative design consistency rating methods for two-lane rural highways.” Draft Final Report, *FHWA-RD-99-172*.

Fitzpatrick, K., Karin, M. B., Anderson, I. B., Collins, J. M., Elefteriadou, L., Paul, G., Douglas, W. H., Nelson, I., Koppa, R., Krammes, R. A., McFadden, J., Parma, K. D., Passetti, K., Poggioli, B., Tsimhoni, O. and Wooldridge M.D. (2000c). “Evaluation of design consistency methods for two-lane rural highways-executive summary.” Draft Final Report, *FHWA-RD-99-173*.

Gangopadhyay. (2010). “Overloading by heavy commercial vehicles on Indian roads-a demon in disguise.” *Indian Highways*, Vol.38, No.11, pp.35-40.

Garber N. J. and Gadiraju, R. (1989). “Factors affecting speed variance and its influence on accidents.” *Transportation Research Record* 1213, Transportation Research Board, National Research Council, Washington D.C., pp. 64-71.

Geometric Design Guide for Canadian Roads (1999),
www.tac-atc.ca/english/bookstore/pdfs/geodes-jan02.pdf

Gibreel, G. M., Easa, S. M., and El-Dimeery, I. A. (2001). “Prediction of operating speed on three-dimensional highway alignments.” *Journal of Transportation Engineering*, Vol. 127, No. 1, January/February,. Paper No. 20084 .ASCE, pp.21-30.

Gibreel, G. M., Easa, S. M., Hassan, Y. and El-Dimeery, I. A. (1999).“State of art of highway geometric design consistency.” *Journal of Transportation Engineering*, Vol. 125, No. 4. July/August, pp. 305-313.

Glennon, J. C., Harwood, D. W., Free, S. and Gray, C. W. (1978). “Highway design consistency and systematic design related to highway safety.” *Transportation Research Record*, No. 681, National Research Council, Washington, D.C., pp. 77-88.

Hassan, Y. (2004). “Highway design consistency – refining the state of knowledge and practice.” TRR, *Journal of the Transportation research board*, No. 1881, National Research Council, Washington D.C, pp. 63-71.

Hassan, Y., Gibreel, G. M. and Easa, S. M. (2000). “Evaluation of highway consistency and Safety practical application.” *Journal of Transportation engineering*, Vol. 126, No. 3, May/June, pp. 193-201.

Hassan, Y., Sayed, T. and Tabernero, V. (2001). “Establishing a practical approach for design consistency evaluation.” *Journal of Transportation Engineering*, Vol. 127, No. 4, Paper No. 22393, July/August, pp. 295–302.

Hirsh, M. (1987). “Probabilistic approach to consistency in geometric design.” *Journal of Transportation Engineering*, 113 (3), pp. 268–276.

Hong, S. J. and Oguchi, T. (2005).“Evaluation of highway geometric design and analysis of actual operating speed.” *Journal of the Eastern Asia Society for Transportation studies*, Vol. 6, pp 1048-1061.

Islam, M. N. and Seneviratne, P. N. (1994). “Evaluation of design consistency of two-lane rural highways.” *ITE Journal*, February 1994, pp. 28–31.

Jessen, D. R., Karen, S. S., Patric, T. M., Geza, P. and Ryan, R. H. (2001). “Operating speed prediction on crest vertical curves of rural two lane highways in nebraska.” *ransportation Research Record* 1751, Paper No.3150, National Research council, Washington, D.C., pp. 67-75.

Joanne, C. and Sayed, T. (2004). “Effect of geometric design consistency on road safety.” *Canadian Journal of Civil Engineering*, Vol. 31, No. 2, pp. 218-227.

Kadilali, L. R. and Shashikala, T. V. (2009). “Road transport demand forecast for 2000 ad revised and demand forecast for 2012.” *Journal of the Indian Roads Congress*, Vol.70-3, pp.235-257.

Kadiyali, L.R., Viswanathan, E., Jain, P.K. and Gupta, R.K. (1981).“Effect of curvature and sight distance on the free speed of vehicles on curves on two lane roads.” *Highway Research Bulletin* (Traffic Engineering), IRC, No.16, pp. 1- 18.

Kanellaidis, G., Golias, J. and Efstathiadis, S. (1990). “Driver’s speed behaviour on rural road curves.” *Traffic Engineering and control*, Vol. 31 No.7/8, pp. 414-415

Karnataka State Report, As on 31-03-2010, http://www.kpwd.gov.in/roads.asp .

Katti, B. K. and Raghavachari, S. (1986).“Modelling of mixed traffic speed data as inputs for the traffic simulation models.” *Highway Research Board* (Traffic Engineering), IRC, Vol. 28, pp. 35-47.

Khanna, S. K. and Justo, C.E.G. (2001). “Highway engineering.” Nem Chand and Bros, Roorkee (U.A).

Korzeniewski, J. (2009). “A report on-rural roads more dangerous than congested urban areas.”
http://www.autoblog.com/2009/10/7/report-rural-roads-more-dangerous-than-congested -urban- areas/, accessed on January, 2012.

Krammes, R. A. (1997). “Interactive highway safety design model: design consistency module.” http://www.tfhrc.gov/pubrds/pr97-10/p47.htm

Krammes, R. A., Hoon, O. H. and Rao, K. S. (1995a). “Highway geometric design consistency evaluation software.” *Transportation Research Record* 1500, Washington, D.C., pp 19–24.

Krammes, R. A., Messer, C. J., Anderson, I. B., Brackett, R. Q., Collins, K., Fink, K., Ottesen, J. L, Pendleton, O. J. and Shafer. M. A. (1995b). “Horizontal alignment design consistency for rural two-lane highways.” *Report FHWA-RD-94-034.* FHWA, U.S. Department of Transportation, Washington, D.C.

Lamm, R., Choueiri, E. M., Hayward, J. C. and Paluri. (1988). “Possible design procedure to promote design consistency in highway geometric design on two lane rural roads.” *Transportation Research Record* 1195, TRB, National Research council, Washington D.C., pp. 111-122

Lamm, R., Chueiri, E. M. and Mailaender.T. (1990). "Comparison of operating speeds on dry and wet pavements of two lane rural highways." *Transportation Research Record* 1280, TRB, National Research council, Washington, D.C., pp. 199-208.

Lamm, R., Choueiri, E.M. and Mailaender, T. (1991)."Side friction demand versus side friction assumed for curve design on two lane rural highways." *Transportation Research Record* 1303, Transportation Research Board, Washington, D.C., pp. 11–21.

Lamm, R., Soilemezoglou, G., Basil, P. and Kanellaidis, G. (1996)."Driving dynamic aspects and related safety issues for modern geometric design of non built up roads." *Transportation Research Record* 1523, TRB, National Research council, Washington D.C., pp. 34-45.

Landge, V. S., Jain, S. S. and Parida, M. (2006). "A critical appraisal of road traffic safety analysis and modeling for non urban highways." *Highway research Bulletin*, IRC, No. 75, Oct, pp. 105-129.

Malhotra, T. K., "Liquor vends on National Highways", http://www.aaui.org/liq.htm

Mawjoud.A.A. and Sofia, G. G. (2008). "Development of models for predicting speed on horizontal curves for two-lane rural highways." *The Arabian Journal for Science and Engineering*, Vol. 33, NO. 2B, October, pp. 365-378.

McCarthy,T. (2011). "Risk assessment review." *European Road Safety Inspection Report*, 25th August, EuRSI D3.2, NRA, Ireland.

McFadden, J. and Elefteriadou, L. (1997). "Formulation and validation of operating speed based design consistency models by bootstrapping." *Transportation Research Record* 1579, Paper No. 787, Transportation Research Board, Washington, D.C., pp. 97–103.

Mcfadden, J. and Elefteriadou, L. (2000)."Evaluating horizontal alignment design consistency of two lane rural highways – development of new procedure." *Transportation Research Record 1737*, Paper No. 0813, pp. 25-32.

Mclean, J. (1981)."Driver speed behavior and rural road alignment design." *Traffic Engineering and control*, Vol.22, No. 4, April, pp. 208-211.

Messer, C. J. (1980). “Methodology for evaluating geometric design consistency.” *Transportation Research Record* 757, National Academy of Sciences, Transportation Research Board, Washington, D.C., pp. 7-14.

Misaghi, P. and Hassan, Y. (2005).“Modeling operating speed and speed differential and two lane rural roads.” *Journal of Transportation Engineering*, Vol.131, No. 6, pp. 408-418

Morrall, J. and Talarico, R. J. (1994). “Side friction demanded and margins of safety on horizontal curves.” *Transportation Research Record*, No.1435, Transportation Research Board, Washington, D.C., pp. 145–152.

MORTH Annual report 2010-11, http://morth.nic.in/writereaddata/mainlinkFile/File414.pdf

Nadaf, A. M. (1999).“Accidents and drivers.” *Indian Journal of transport management*, Vol.23, No.6, pp. 361-369.

Nagaraj, B. N., George, K. J. and Velkennedy, R. (1990). “Exploration in the use of Speed profiles for Identification of Accident Prone Locations.” *Highway Research Bulletin, Traffic Engineering*, IRC, No. 42, pp. 137-163.

Nicholson, A. (1998). “Super elevation, side friction and roadway consistency.” *Journal of Transportation Engineering*, Vol. 124, No. 5, pp. 411-418.

Ottesen, L. J. and Krammes, R. A. (2000).“Speed profile model for a design consistency evaluation procedure in the United States.” *Transportation Research Record* 1701, Paper No. 325, TRB, Washington D.C., pp. 76-85.

Park, Y. J. and Saccomanno, F. F. (2006).“Evaluating speed consistency between successive elements of a two lane rural highway.” *Journal of the Transportation Research Board*- Part A, No.40,TRB, Washington D.C., pp. 375–385.

Partha, P. D., Satish C. and Gangopadyaya, S. (2008). “Speed studies on two lane Indian highways.” *Indian Highways*, Vol.36, No.6, pp. 9-19.

Perco, P. (2006). “Desirable length of spiral curves for two lane rural roads.” *Journal* *of the Transportation Research Board*, No. 1961, TRB, Washington D.C., pp. 1-8.

Poe, M., Christophar.and Mason, J. M. (1995).“Geometric design guidelines to achieve desired operation speed on Urban streets.” *Institute of Transportation Engineers 65th Annual meeting*, Compendium of Technical Papers., pp. 70-74.

Polus, A .and Dagan, D. (1987). “Models for evaluating the consistency of highway alignment.” *Transportation Research Record*, No.1122, Transportation Research Board, Washington, D.C., pp. 47-56.

Polus, A. (1980).“The Relationship of overall geometric characteristics to the safety level of rural highways.” *Traffic Quarterly*. Vol. 34, No. 4, October, pp. 575-585.

Polus, A., Fitzpatrick, K. and Fambro, D. (2000). “Predicting operating speeds on tangent sections of two-lane rural highways.” *Transportation Research Record* 1737, Transportation Research Board, National Research Council, Washington, D.C., pp. 50–57.

Rajan, I. (2006). “Rescue operation of accident victims – A review.” *Indian Highways*, Vol.34, No.3, pp. 5-10.

Road transport year book (2006-07), *Transport Research Wing, Ministry of Shipping, Road Transport & Highways*, Government of India, New Delhi, March, 2009.

Road Transport Year Book (2007-2009) (Volume - I) *Transport Research Wing, Ministry of Shipping, Road Transport & Highways*, Government of India, New Delhi, (www.morth.nic.in/writereaddata/mainlinkfile/file420.pdf).

Satish, C. and Prashanth, K. B. (2004). “Accident analysis on two lane roads.” *Highway Research Board*, No. 70, IRC, November, pp. 77-92.

Shaheem, G. S. and Gupta Das. C. (2005). “Impact of road development on road safety – A case study of Aluva – Cherthala section of NH 47.” *Journal of the Indian road congress*, October, Vol. 66-3, pp. 615-639.

Sikdar, P. K. (2005).“Road safety in India.” *Indian Highways*, Vol.33, No.11, November , pp. 91-99.

Srinivasan, N.S. (2011). “Critical review of thrust areas in road safety action plan” *Indian Highways*. Vol.39, No.12, pp. 9-21.

Stewart, D. and Christoper, J. C. (1990).“A remedy for accidents at bends.” *Traffic engineering and control*, Feb, pp. 87-89.

Taylor, M. (2001). “The speeds of vehicles which are involved in Fatal accidents.” *Traffic engineering and control*, Feb, pp. 44-48.

Urban Transport development in India Energy and Infrastructure Unit South Asia Region March 2005. http://siteresources.worldbank.org/INTSARREGTOPTRANSPORT/Resources/UrbanTransportSectorStrategyNote.pdf accessed on 24 dec2011

Veith, G. and Truner, B. (2006). “The effect of geometric road design standards on road safety.” *Road safety Risk Reporter*, No. 4, August, pp. 1-3.

Vivian, R. R. (2006) “Studies on road safety problems under mixed traffic flow” a PhD Thesis submitted to Bangalore University (Unpublished).

Washington, S., Persaud, B., Lyon, C. and Oh, J. (2005).“Validation of accident models for intersections.” Publication No. *FHWA-RD-03-037*, Federal Highway Administration Washington, DC.

Wooldridge, M. D., Fitzpatrick, K., Douglas, W. H., Anderson, I. B. and Elefteriadou, P. L. (2003). “Geometric design consistency on high speed rural two lane roadways.” *NCHRP 502*, TRB, Washington. D.C.

Wooldridge, M. D., Fitzpatrick, K., Koppa, R. and Bauer, K. (2000a).“Effects of horizontal curvature on driver visual demand.” *Transportation Research Record* 1737, Transportation Research Board, Washington, D.C., pp. 71-77.

Wooldridge, M. D., Green, P., Bauer, K. and Fitzpatrick, K. (2000b).“Comparison of driver visual demand in test track, simulator and on road environments.” *Proceedings of the 79th Transportation Research Board Annual Conference*, Washington D.C., January 2000.

Xiong, J., Huasen, W. and Fengxiang, G. (2005). "Freeway design consistency evaluation - a case study on driving simulator" ICTCT Ex ICTCT Extra Workshop, SESSION 6: Highway problems and Accident Analysis, Beijing, pp. 231-242.

Zegeer, C. V., Hummer, J., Herf, L., Reinfurt, D. and Hunter, W. (1986). "Safety effects of cross-section design for two- lane roads." Report No. *FHWA-RD-87-008*, Federal Highway Administration, Washington, D. C.

APPENDIX-1

DATA COLLECTED

Table A1.1 Geometric, Speed and accident details of selected Horizontal curves.

Curve	R_c (m)	Δ (Deg)	L_H (m)	PTL (m)	PTLS (m)	Superelevation			Sight distance in m				Road width(m)			Shoulder width(m)			Operating speed in km/h				Speed differential km/h	Number of accidents		
						e_S (%)	e_M (%)	e_E (%)	SD_{TS}	SD_{TM}	SD_S	SD_M	W_S	W_M	W_E	S_S	S_M	S_E	VTM	VS	VM	VE	$(\Delta V)_{85TMM}$	Fatal	Grevious	Simple
H1	219	40	153	110	55	1.2	5.17	1.43	93	85	50	72	5.6	5.8	5.6	2	2.5	2	62.0	52.0	39.0	46.5	27.0	1	2	2
H2	181	60	190	112	55	2.78	3.57	2.18	80	60	90	80	5.4	5.5	5.6	1.5	1.5	1.2	55.5	44.0	40.0	41.5	28.0	-	1	
H3	176	56	174	230	115	1.44	5.45	2.12	95	73	71	69	5.5	5.5	5.2	2	2.2	2	66.0	54.5	52.5	53.5	24.0	-	5	1
H4	115	63	127	230	115	1.55	4.68	2.12	120	100	36	38	5.1	5.5	5.2	1.4	1.7	1.5	66.0	50.0	52.0	55	26.0	-	-	2
H5	256	33	149	100	50	1.02	1.98	1.71	90	70	50	38	5.4	5.6	5.5	1	2.1	2	65.5	59.0	51.0	59.5	23.0	-	-	-
H6	56	87	85	200	100	2.59	5.69	2.53	103	80	42	38	5.2	5.7	5.6	2.3	2.4	1.4	64.0	50.0	40.0	57	26.0	-	1	-
H7	160	41	114	120	60	1.34	4.72	1.32	97	76	55	81	5.8	5.8	5.2	2	2.2	2.1	64.0	55.5	42.0	54.5	27.0	-	-	-
H8	145	41	103	100	50	1.29	2.05	1.03	87	80	72	70	5.1	5.5	5.3	1.9	1.7	1.8	53.0	53.0	43.0	53	29.0	-	1	-
H9	168	44	128	110	55	1.11	2.73	1.23	91	75	50	70	5.9	5.5	5.8	1.9	1.9	1.7	62.0	53.5	48.0	53	21.0	-	-	-
H10	368	16	100	360	180	1.36	4.14	1.11	110	100	59	67	5.5	5.6	5.4	2	2.2	2.1	67.0	59.0	51.0	57	27.0	1	1	-
H11	300	14	75	250	125	1.19	4.44	1.93	116	100	63	60	5.6	5.6	5.3	1.8	1.9	1.5	66.0	56.0	46.5	53.5	24.0	-	-	2
H12	188	26	85	120	60	1.88	3.92	1.11	80	80	35	57	5.5	5.5	5.4	2	2.3	1.7	59.5	52.0	41.0	50	25.0	-	1	-
H13	148	34	87	100	50	1.94	4.01	2.01	92	65	47	41	5.6	5.6	5.5	2	2.1	1.6	60.0	51.0	42.0	51	17.5	-	1	-
H14	150	42	111	160	80	1.4	2.36	1.02	90	72	45	56	5.6	5.8	5.7	1.7	2.2	1.6	67.0	53.0	44.0	50	30.0	-	-	1
H15	105	58	107	160	80	1.14	3.01	1.19	80	80	83	39	4.8	4.9	4.4	1.6	1.7	1.8	57.0	46.0	41.0	53.5	29.0	-	1	-
H16	70	53	65	100	50	2.13	6.02	3.64	90	84	46	46	5	5.5	5.4	1.4	1.8	1.7	66.0	55.0	46.5	55	26.0	-	-	-

H17	117	60	122	180	90	1.67	4.03	3.06	100	82	55	67	4.7	5.8	5.5	1.9	2.2	1.7	67.0	49.0	38.0	48	31.0	-	-	1
H18	83	99	143	120	60	1.13	6.42	3.91	70	55	37	35	5.4	5.5	5.1	1.7	1.8	1.7	66.0	54.0	46.0	52	23.5	-	-	-
H19	102	65	116	230	115	1.2	6.14	2.41	110	100	36	42	5.3	5.6	5.4	2.1	2.4	2	70.0	56.0	42.5	50.5	30.0	-	-	-
H20	256	40	180	230	115	1.37	4.2	1.4	105	90	52	41	5	5.7	5.4	1.6	2.4	2.2	69.5	57.5	42.5	54	32.0	-	1	-
H21	223	23	91	100	50	5.05	5.92	5	65	55	45	36	5.1	5.6	5	1.8	2	2	55.0	48.0	40.0	45.5	23.0	-	-	-
H22	169	46	137	110	50	3.06	4.42	1.35	70	65	45	62	5.3	5.5	5	1.7	1.9	1.8	67.5	58.5	47.0	57.5	27.0	-	-	-
H23	165	42	122	150	70	2.4	3	1.89	85	70	40	40	4.9	4.7	5.2	1.6	1.9	1.7	56.0	51.5	42.5	53.5	30.0	-	1	1
H24	185	67	215	150	70	0.86	6.85	1.51	90	70	51	48	5	5.5	5.3	2	1.5	2.2	59.0	58.5	50.5	57	26.0	-	1	-
H25	180	14	45	100	50	2	6.61	1.18	75	63	60	63	5.8	5.8	5.3	2.2	2.4	2.1	60.5	51.5	40.0	51	25.0	-	-	-
H26	195	65	220	120	60	1.07	6.3	5.8	70	65	42	49	5	5.6	5	2.3	2.5	2	65.0	55.0	46.0	53	24.0	1	1	-
H27	155	68	185	140	70	1.92	6.21	1.29	90	85	57	55	5.6	5.6	5	1.7	2.4	1.8	64.0	57.5	48.0	53	25.0	-	1	-
H28	130	81	184	150	75	1.11	1.69	1.04	81	75	62	42	5.2	5.5	5.3	1.6	1.9	1.7	65.0	54.5	43.2	55.5	27.0	-	1	-
H29	228	56	224	140	65	1.25	2.01	1.41	90	80	62	45	5.6	5.5	5.4	2	1.6	0.9	70.0	63.5	55.0	60.5	24.0	-	-	-
H30	291	32	163	105	50	3.42	5.18	3.57	68	42	45	40	5.6	5.6	5.6	1	0.7	1.5	56.0	57.5	56.0	61.5	21.0	-	-	-
H31	121	81	172	255	125	2.05	6.21	4.13	126	100	65	70	4.4	5	5.3	1.2	1.2	1.1	70.0	50.0	39.0	53	35.0	1	1	2
H32	635	15	170	255	125	1.5	2.01	1.12	113	105	63	62	5.4	5.4	5.4	2.1	2.6	2.2	72.0	58.5	52.5	55.5	32.5	1	1	1
H33	45	104	82	350	160	1.11	6.13	1.43	110	90	55	64	6.3	6.5	5.6	3.3	3.2	2.2	64.0	43.5	38.5	43	29.0	-	1	1
H34	297	51	263	165	80	1.12	5.25	1.42	75	72	40	40	5.3	6.1	5.6	2.4	2.3	2.1	64.0	62.0	48.0	52.5	27.0	-	-	-
H35	86.5	7	105	120	60	0.73	6.43	3.16	70	62	40	45	5.5	5.7	5.7	2	2.3	2.4	63.0	47.0	40.0	45	26.0	-	1	-
H36	47	110	90	100	50	1.14	3.12	2.51	80	75	36	42	5.3	5.7	5.4	2.1	2.2	2.2	62.0	54.0	50.0	48	23.0	1	-	1
H37	426	35	229	152	120	6.42	4.84	5.09	53	65	50	60	5.45	6.6	5.1	2.2	1.7	1.8	69.0	60.0	56.0	62.5	22.0	-	-	-
H38	188	23	75	100	50	1.75	4.82	0.37	80	70	60	65	5.7	5.6	5.4	2.3	1.5	1.2	68.5	58.0	47.5	57.5	26.0	-	-	-
H39	171	37	110	126	60	4.52	3.92	4.46	70	65	40	45	5.3	5.1	5.6	0.8	2	1	60.0	58.5	52.0	58	15.0	-	-	-
H40	305	30	161	264	120	1.13	11.3	0.57	118	98	56	60	5.3	4.8	5.3	2	1.3	0.5	65.0	60.0	54.5	56	24.0	-	-	-
H41	125	56	122	104	50	1.48	11.1	2.45	60	56	45	50	5.4	5	5.3	2.8	2	1.6	57.0	47.0	46.0	50	20.0	-	1	-
H42	88	79	121	104	50	0.37	7.4	4.33	79	70	60	55	5.3	5.5	5.3	2	1.4	2.3	64.0	57.0	53.5	58.5	18.0	1	1	2
H43	106	32	60	170	85	1.82	6.63	13.2	120	72	35	66	5.2	5.5	5.3	1.7	1.5	1.2	64.0	55.0	45.0	58	27.0	-	1	-

H44	88	64	98.5	105	50	0.54	8.5	0.75	74	68	61	53	5.5	5.5	5.3	1	1	1.4	63.0	57.0	49.0	54	20.0	1	1	-
H45	128	25	56	140	70	1.27	5.27	5.45	80	60	53	52	5.5	5.5	5.5	0.9	1.5	1.3	63.0	57.0	48.0	55.5	20.0	-	1	1
H46	254	41	180	300	150	3.63	5.09	5.45	110	80	35	77	5.5	5.5	5.5	0.6	1	1.3	65.5	54.0	44.0	55	27.0	-	-	-
H47	114	69	137	120	60	0	5.09	0.36	80	74	65	69	5.1	5.5	5.5	0.8	1	1.6	60.0	55.0	49.5	54.5	19.0	-	-	-
H48	153	36	96	175	85	1.45	8.58	3.39	90	75	70	65	5.5	5.5	5.3	1.3	1.3	1	62.0	55.5	52.0	55	16.0	-	1	-
H49	237	18	75	100	50	4.45	5.45	1.42	95	70	60	75	5.5	5.5	5.6	1.5	0.6	1.3	66.5	59.5	56.0	58	22.0	-	-	1
H50	120	34	71	120	60	1.32	13.3	4.63	70	53	126	75	5.4	5.5	5.5	1.3	0.9	1.3	59.0	53.0	49.0	54.5	16.0	-	-	-
H51	58	49	50	175	85	0.18	2.16	0.71	95.0	80.0	34.0	36.0	5.65	5.55	5.65	3.5	1.5	1.0	68.0	56.0	39.0	45.5	32.0	-	-	1
H52	209	18	65	175	85	1.61	3.18	3.75	65.0	50.0	40.0	36.8	5.60	5.50	5.60	3.4	4.1	4.9	60.0	48.0	40.0	46	23.0	-	-	-
H53	87	56	86	423	210	2.91	3.96	1.89	100	92.0	50.3	57.6	5.50	5.55	5.55	4.9	7.7	6.0	73.0	60.5	48.0	58.5	29.0	-	-	-
H54	126	32	71	423	210	1.95	5.18	1.94	100	80.0	63.0	71.0	5.65	5.60	5.45	4.5	2.7	2.5	66.5	57.0	47.5	56.5	23.0	-	-	1
H55	70	48	59	144	70	1.49	4.59	2.19	77.0	74.0	52.0	66.0	5.70	5.55	5.70	5.0	2.6	2.9	67.0	61.0	46.0	40	27.0	-	-	-
H56	180	30	94	144	70	2.14	1.48	2.14	73.0	70.0	43.0	49.0	5.85	5.73	5.85	2.9	3.0	2.8	65.0	51.5	44.5	54	23.0	-	-	-
H57	180	22	70	180	90	1.98	4.18	0.54	80.0	70.0	31.0	44.0	5.55	5.50	5.55	2.8	3.6	3.5	63.0	57.0	45.0	55	25.0	-	-	-
H58	106	33	61	185	90	2.21	2.07	3.65	60.0	51.0	30.0	45.0	5.65	5.55	5.75	2.4	2.1	2.0	60.0	52.0	41.0	59.5	23.0	-	-	-
H59	104	36	65	180	90	3.57	6.61	1.61	70.0	52.0	39.0	48.0	5.60	5.65	5.60	2.3	2.6	3.3	65.0	58.5	46.0	56	24.0	-	-	-
H60	78	37	50	180	90	0.80	5.36	1.75	70.0	50.0	30.0	47.0	5.60	5.60	5.70	2.8	2.7	3.6	56.0	48.0	44.0	47	15.0	-	-	-
H61	42	70	51	169	80	0.59	6.23	2.57	67.0	59.0	50.0	70.0	5.10	6.10	5.45	3.8	3.2	3.3	65.5	60.0	53.5	59	18.0	-	-	-
H62	259	22	100	169	80	10.2	0.84	2.26	93.0	73.0	40.0	42.0	5.90	5.95	5.75	2.7	2.6	3.0	68.0	58.0	50.0	58	23.0	-	-	-
H63	185	40	130	258	125	7.08	4.62	4.39	107	90.0	42.0	83.0	5.65	5.85	5.70	1.9	3.8	4.3	64.0	48.0	44.0	47.5	27.0	-	-	-
H64	217	45	169	244	120	2.18	2.55	3.75	83.0	73.0	43.0	49.0	5.50	5.50	5.60	2.5	2.0	2.0	61.5	50.0	45.5	48.5	24.0	-	-	-
H65	157	33	90	244	120	1.75	4.09	4.20	90.0	85.0	45.0	64.0	5.70	5.75	5.60	3.1	3.1	2.3	61.0	55.0	50.0	53	20.0	-	-	-
H66	262	17	78	150	125	9.09	1.74	1.81	66.0	61.0	51.0	64.0	5.50	5.75	5.53	2.6	3.0	2.5	68.0	55.0	46.0	53.5	16.0	-	-	-
H67	84	59	86	141	70	11.6	5.83	1.87	94.0	78.0	37.0	45.0	5.60	5.66	5.62	6.5	3.8	1.9	58.0	51.5	38.0	47.5	25.5	-	-	1
H68	324	18	100	125	60	2.50	2.52	1.44	90.0	66.0	44.0	47.0	5.60	5.55	5.55	2.5	2.5	2.8	56.0	52.0	45.0	49	18.0	-	1	2
H69	163	30	85	155	80	4.40	2.63	2.67	99.0	70.0	50.0	66.0	5.68	5.70	5.62	3.4	4.1	3.9	69.0	59.0	51.5	56	20.5	-	2	2
H70	123	54	116	155	80	3.15	1.75	8.77	80.0	65.0	33.0	46.0	5.55	5.70	5.70	3.7	3.4	3.5	59.5	54.5	44.0	48	23.0	-	-	-

H71	101	34	61	133	65	3.13	2.70	0.90	80.0	63.0	41.0	82.0	5.60	5.55	5.55	3.0	2.5	3.0	66.0	59.0	52.0	58	22.0	-	1	-
H72	337	18	103	133	65	1.07	2.87	0.62	77.0	72.0	39.0	44.0	5.60	5.75	5.62	1.5	0.7	2.0	66.5	56.5	52.0	58.5	18.5	-	-	-
H73	208	25	90	218	100	1.26	9.49	1.83	79.0	76.0	35.0	45.0	5.15	5.27	5.45	3.5	2.3	0.8	70.0	65.5	56.0	62	21.0	-	-	-
H74	313	15	80	175	85	2.50	2.52	1.44	71.0	66.0	34.0	39.0	5.60	5.55	5.55	2.5	2.5	2.8	69.0	62.5	54.0	58	23.0	-	-	-
H75	117	46	94	104	50	2.65	5.18	4.73	65.0	52.0	30.0	34.0	5.65	5.60	5.50	0.7	1.2	0.6	60.5	52.5	48.0	52	19.5	-	1	-
H76	250	18	79	108	50	1.27	3.33	3.68	61.0	60.0	40.0	54.0	5.50	6.00	6.80	0.5	0.5	1.0	67.0	58.0	52.0	57.5	19.5	-	-	-
H77	170	34	100	120	60	2.86	0.14	0.57	37.0	32.0	19.0	27.0	7.00	7.00	7.00	1.2	0.8	0.3	65.5	54.5	47.0	55	22.0	-	1	-
H78	81	37	52	118	60	2.94	3.52	3.57	62.0	57.0	28.0	50.0	6.80	7.10	7.00	1.5	0.8	1.5	66.5	52.0	44.5	52.5	26.0	-	-	-
H79	79	47	64	118	60	5.00	2.95	2.46	84.0	52.0	36.0	28.0	7.00	6.10	7.10	0.6	2.0	1.4	66.5	56.0	50.0	55	18.5	-	-	-
H80	109	35	66	100	50	0.09	5.78	5.41	60.0	53.0	25.0	36.0	5.60	5.80	5.55	0.3	0.4	0.7	59.5	47.0	39.5	50	22.0	-	2	-
H81	255	13	60	125	60	0.27	3.42	3.25	60.0	45.0	46.0	50.0	5.50	5.84	5.70	0.2	0.9	0.2	57.0	53.5	43.5	48.5	24.0	-	-	1
H82	45	103	81	175	80	2.30	5.82	0.55	80.0	70.0	23.0	29.0	5.65	5.50	5.45	1.0	1.4	1.0	69.0	65.0	66	62	16.5	-	1	-
H83	144	32	81	146	70	3.00	2.81	0.49	65.0	60.0	40.0	29.0	6.00	6.40	6.10	0.5	0.2	0.5	60.0	47.0	39.0	44.5	30.0	1	-	-
H84	392	15	100	146	70	2.12	1.30	0.83	65.0	55.0	40.0	35.0	5.65	5.60	5.40	0.2	0.3	0.4	61.5	57.5	51.5	55.5	12.0	-	-	-
H85	119	42	87	100	50	0.92	2.68	3.89	53.0	50.0	26.0	18.0	5.42	5.60	5.14	0.4	0.1	0.9	58.0	52.0	44.0	50.5	22.0	-	-	-
H86	273	17	80	185	90	2.73	0.67	1.12	70.0	41.0	32.0	39.0	5.50	5.20	4.48	0.2	0.2	0.1	63.5	57.0	48.0	57.5	18.0	-	-	-
H87	112	30	58	120	60	2.60	2.39	1.61	53.0	52.0	39.0	27.0	5.77	5.65	5.60	1.5	0.2	1.3	59.0	54.0	47.0	53	19.5	-	-	-
H88	90	38	60	120	60	1.27	2.45	2.18	60.0	55.0	35.0	35.0	5.50	5.50	5.50	1.0	1.0	0.9	60.0	54.5	45.0	51	25.5	-	1	-
H89	279	25	120	170	85	2.30	5.82	0.55	99.0	72.0	37.0	39.0	5.65	5.50	5.45	1.0	1.4	1.0	67.5	58.0	50.5	56	22.0	-	-	-
H90	263	17	80	140	70	1.13	3.39	2.00	66.0	52.0	50.0	53.0	5.30	5.60	5.50	1.8	1.7	1.8	58.5	49.5	46.0	51	15.0	-	1	-
H91	141	37	90	170	80	2.75	3.84	1.79	66.0	64.0	41.0	58.0	5.28	5.34	5.58	1.2	1.0	0.6	65.0	59.0	52.0	60.5	19.0	-	-	-
H92	384	13	90	110	55	3.93	3.42	1.65	110	86.0	36.0	38.0	5.35	5.55	5.45	0.9	2.0	1.3	64.5	56.0	46.0	54.5	24.5	-	-	-
H93	301	23	122	118	55	0.56	0.46	0.45	100	63.0	43.0	47.0	5.40	5.55	5.50	1.1	1.4	1.5	67.0	58.5	54.0	58.5	16.0	-	-	-
H94	503	17	150	405	200	4.00	6.94	3.02	107	100.0	49.0	54.0	5.50	5.50	5.30	0.6	1.9	2.0	68.0	61.5	55.0	61	20.0	-	-	-
H95	68	55	65	185	90	1.50	2.13	0.44	62.0	64.0	35.0	36.0	5.00	5.60	5.65	0.5	1.2	1.5	65.0	50.0	42.0	47	28.5	1	2	-
H96	82	32	45	100	50	2.12	3.27	2.73	56.0	47.0	28.0	38.0	5.20	5.35	5.50	1.9	1.3	1.3	57.0	48.5	42.0	47	18.5	-	-	-
H97	294	20	105	198	95	0.53	5.21	2.46	58.0	57.0	37.0	39.0	5.68	5.85	5.70	3.1	2.2	2.6	58.5	59.0	46.5	51.5	16.0	-	-	-

H98	126	43	93	198	95	3.21	3.58	1.92	80.0	68.0	52.0	55.0	5.30	5.30	5.20	1.5	1.4	1.6	64.0	56.0	47.0	55	20.5	-	-	-
H99	119	34	70	170	85	1.21	4.99	0.09	60.0	55.0	28.0	38.0	5.37	5.61	5.30	4.8	6.4	5.3	61.0	51.0	46.0	52	18.0	-	3	-
H100	114	57	113	228	110	0.09	5.25	4.45	48.0	43.0	34.0	30.0	5.50	5.90	5.95	2.6	2.7	2.0	58.0	50.5	46.0	52.5	19.0	-	-	1
H101	210	41	150	215	100	3.68	5.53	1.88	52.0	54.0	39.0	44.0	5.70	5.55	5.85	3.0	4.0	1.6	67.2	59.0	53.0	58.5	22.0	-	-	-
H102	342	28	170	210	105	4.31	6.21	1.82	100	75.0	42.0	45.0	5.80	5.80	5.50	1.2	1.3	1.4	67.0	57.0	52.0	56	22.0	-	1	2
H103	210	26	95	210	105	0.90	2.00	2.20	60.0	51.0	45.0	45.0	5.55	5.50	5.45	1.6	2.3	2.6	66.0	57.5	52.0	57.5	23.0	-	-	-
H104	330	21	120	308	150	0.86	3.39	2.89	59.0	66.0	40.0	45.0	5.80	6.20	5.70	2.2	3.0	2.8	65.5	56.5	51.0	57	23.0	-	-	-
H105	370	19	125	225	110	2.18	6.52	0.56	100	80.0	36.0	39.0	5.50	5.60	5.40	2.5	1.7	2.0	68.0	54.0	44.0	44	26.0	-	1	-
H106	140	29	72	208	105	2.83	5.17	1.54	84.0	56.0	58.5	46.0	5.30	5.80	5.20	3.0	1.9	1.6	66.0	55.5	44.0	54.5	27.0	-	-	-
H107	89	52	80	175	85	1.31	9.34	8.89	52.0	48.0	30.0	47.0	5.35	5.50	5.40	1.2	2.5	1.7	59.5	55.0	48.5	53	15.5	-	-	-
H108	235	20	80	175	85	5.09	6.17	3.02	50.0	45.0	36.0	44.0	5.30	4.78	5.30	3.0	1.8	2.4	65.0	53.0	45.0	53	22.0	-	-	-
H109	78	34	46	100	50	4.74	6.31	3.00	53.0	44.0	30.0	35.0	4.85	5.55	5.50	2.2	2.0	1.9	56.0	47.0	40.0	45	16.5	-	2	1
H110	193	23	78	100	50	4.55	4.04	2.91	61.0	52.0	40.0	50.0	5.50	5.70	5.50	1.1	1.3	0.7	57.5	53.0	42.5	52.5	19.5	-	-	-
H111	219	18	67	100	50	2.99	8.18	4.31	57.0	49.0	41.0	45.0	5.35	5.50	5.10	1.7	1.3	1.8	58.5	50.5	45.0	50.5	16.0	-	2	-
H112	65	53	60	146	70	1.70	5.18	3.87	50.0	45.0	31.0	40.0	5.60	5.60	5.43	1.9	1.8	0.4	56.0	48.5	43.5	48	16.0	-	-	-
H113	72	34	43	163	80	1.20	6.13	4.62	70.0	50.0	32.0	43.0	5.00	5.55	5.20	2.4	2.8	2.8	64.0	53.0	46.0	53.5	22.5	-	-	-
H114	64	43	48	163	80	0.09	5.25	4.45	52.0	50.0	37.0	30.0	5.50	5.90	5.95	2.6	2.7	2.0	64.0	51.5	45.5	51	22.0	-	1	-
H115	210	22	80	125	60	0.91	3.21	0.46	50.0	45.0	30.0	26.0	5.50	5.30	5.40	0.3	0.3	0.1	60.0	53.0	44.0	52	21.0	-	-	-
H116	93	40	65	270	135	1.81	3.00	5.23	88.0	115.0	42.0	32.0	5.25	5.50	5.45	0.5	0.5	0.5	66.0	47.5	41.5	45.5	31.5	-	2	-
H117	475	21	175	270	135	0.78	2.43	2.88	105	95.0	39.0	60.0	5.10	5.35	5.20	0.1	0.2	0.1	64.0	56.0	52.5	55	20.0	-	2	1
H118	32	74	42	103	50	3.70	9.83	0.10	60.0	55.0	25.0	31.0	5.00	5.80	5.10	0.3	0.5	0.1	66.5	49.0	38.0	42.5	31.0	-	2	1
H119	99	32	55	103	50	1.30	4.23	3.77	47.0	40.0	30.0	32.0	5.40	5.20	5.30	0.3	0.3	0.3	61.0	51.5	40.0	50	27.0	-	-	-
H120	149	40	104	162	80	7.36	7.67	3.00	68.0	46.0	30.0	35.0	5.30	6.00	5.50	0.1	0.5	0.3	66.0	50.5	45.5	50	22.0	-	1	-
H121	400	24	170	162	80	0.82	2.07	0.09	80.0	67.0	23.0	28.0	5.50	5.80	5.30	0.8	1.0	0.5	67.5	60.0	51.0	58	20.5	-	1	-
H122	313	27	115	225	110	2.31	2.74	0.20	90.0	120.0	70.0	65.0	5.40	5.50	5.00	1.6	1.2	1.3	68.5	55.0	45.0	51	29.0	-	1	1
H123	283	34	170	175	85	0.58	1.11	0.69	95.0	70.0	60.0	75.0	5.20	5.50	5.10	3.0	3.0	0.8	53.0	48.0	42.0	40	17.0	1	-	-
H124	59	59	61	173	85	1.30	2.98	1.57	78.0	50.0	36.0	40.0	5.40	5.70	5.40	1.0	1.2	0.4	61.0	52.0	45.5	50.5	21.0	-	-	-

H125	102	38	67	173	85	2.79	4.73	0.45	56.0	50.0	30.0	20.0	5.20	5.60	5.50	0.2	2.0	2.0	59.5	45.5	37.0	41.5	28.5		1	-
H126	65	48	55	133	65	1.79	3.06	0.10	60.0	54.0	32.0	36.0	5.30	5.50	5.15	0.1	0.5	0.8	55.0	46.0	40.5	45	18.0	-	-	-
H127	86	42	63	160	80	1.45	3.09	0.74	65.0	61.0	30.0	40.0	5.50	5.50	5.40	0.4	0.3	0.3	56.5	47.0	37.5	48	23.0	1	1	-
H128	76	46	62	160	80	0.28	6.55	2.69	70.0	65.0	29.0	32.0	5.40	5.50	5.40	0.6	0.9	0.7	58.5	50.5	44.0	48	18.5		-	-
H129	51	62	55	175	85	2.02	4.75	4.46	60.0	55.0	25.0	43.0	5.20	5.90	5.60	0.6	0.9	0.5	56.5	47.5	39.5	45	21.0	-	1	
H130	116	33	67	175	85	2.17	3.82	2.07	60.0	42.0	28.0	35.0	5.30	5.50	5.50	0.0	0.1	0.5	59.0	51.0	42.5	46	22.0	-	-	-
H131	57	50	50	115	55	4.04	6.96	0.94	70.0	60.0	35.0	38.0	5.70	5.75	5.30	0.4	0.2	0.8	58.0	50.5	46.0	48.5	15.0	-	-	-
H132	29	100	51	200	100	0.28	7.14	0.37	50.0	43.0	20.0	37.0	5.30	6.30	5.45	0.1	0.2	0.9	49.0	35.5	30.0	33	25.0	-	1	1
H133	120	48	100	200	100	3.90	6.00	0.50	63.0	43.0	31.0	35.0	5.00	5.50	5.00	0.1	0.3	0.1	47.0	41.0	38.0	40.5	14.5	-	-	-
H134	120	21	45	115	55	0.09	6.96	3.15	50.0	42.0	30.0	40.0	5.50	5.60	5.40	0.3	0.8	0.3	57.5	52.0	46.0	50.2	16.5	-	1	-
H135	128	29	65	115	55	0.93	4.55	0.74	52.0	50.0	32.0	37.0	5.40	5.50	5.40	0.5	0.3	0.5	55.0	47.5	39.0	45.5	20.0	-	1	-
H136	112	24	46	100	50	5.38	7.98	1.98	79.0	70.0	60.0	55.0	5.30	5.55	5.30	0.9	1.3	1.0	64.0	56.5	50.0	56	21.5	-	-	-
H137	221	31	120	125	55	0.09	3.21	1.30	100	72.0	35.0	66.0	5.50	5.60	5.40	1.2	0.8	0.5	67.5	65.5	55.0	59.5	21.0	-	-	1
H138	278	27	130	150	75	0.91	5.41	1.27	65.0	60.0	33.0	51.0	5.50	5.55	5.50	0.5	0.8	1.0	66.0	54.0	49.0	56	18.0	-	-	-
H139	274	26	123	300	150	0.73	1.25	1.51	120	70.0	55.0	70.0	5.50	5.60	5.30	0.5	0.6	1.2	70.0	60.0	52.0	60	22.0	-	-	-
H140	153	72	193	300	150	0.80	4.30	4.42	120	100.0	83.0	72.0	5.60	5.70	5.65	0.4	0.5	0.5	68.0	57.5	51.5	56.5	25.5	-	-	-
H141	128	58	130	225	110	2.18	2.55	3.75	90.0	65.0	48.0	42.0	5.50	5.50	5.60	2.5	2.0	2.0	63.0	52.0	45.0	54.5	24.0	-	-	-
H142	74	47	60	155	75	0.93	4.12	0.36	74.0	66.0	62.0	47.0	5.40	5.70	5.50	0.7	1.5	0.2	63.5	50.5	44.0	51	24.5	-	-	-
H143	77	49	66	155	75	0.36	3.73	3.27	120	80.0	32.0	56.0	5.50	5.50	5.50	1.5	2.0	0.8	66.0	55.0	52.0	55.5	21.0	-	2	-
H144	215	24	89	150	75	3.82	4.21	2.86	95.0	80.0	41.0	51.0	5.50	5.70	5.60	0.4	0.6	0.3	72.0	60.0	50.5	59	25.5	-	1	-
H145	200	25	87	150	75	2.14	7.14	2.86	89.0	84.0	30.0	83.0	5.60	5.60	5.60	0.3	0.8	0.5	64.0	56.5	46.0	65	21.5	-	1	-
H146	49	48	41	208	100	3.96	9.64	1.85	101	98.0	31.0	37.0	5.30	5.50	5.40	0.2	0.5	0.8	65.0	56.0	44.0	50.5	27.0	-	-	-
H147	108	42	80	102	50	1.70	8.07	5.71	65.0	44.0	40.0	24.0	5.60	5.70	5.60	0.2	0.3	0.4	62.5	54.0	49.0	56	21.0	-	-	-
H148	77	27	36	102	50	0.71	5.26	4.81	55.0	53.0	44.0	45.0	5.60	5.70	5.20	1.0	0.9	0.7	61.0	53.0	48.0	53.5	21.0	-	-	-
H149	121	26	55	115	55	2.55	5.27	3.33	65.0	60.0	38.0	48.0	5.50	5.60	5.40	1.2	0.4	0.4	61.0	55.5	47.0	52.5	19.0	-	-	-
H150	597	11	115	110	55	1.30	1.57	2.04	80.0	74.0	37.0	72.0	5.40	5.50	5.40	1.2	0.9	0.4	62.0	55.5	50.0	54.5	18.0	1	2	-
H151	273	17	80	125	60	3.27	4.21	8.04	90.0	80.0	38.0	74.0	5.50	5.70	5.60	0.2	0.2	0.2	65.0	58.0	51.0	56	21.0	1	1	-

H152	366	14	87	125	60	0.90	3.16	1.43	59.0	58.0	39.0	56.0	5.55	5.70	5.60	0.1	0.7	0.9	66.0	62.5	58.0	60.5	18.0	-	-	-
H153	400	12	131	284	140	0.73	2.46	0.96	108	95.0	45.0	55.0	5.50	5.70	5.70	0.5	0.5	0.4	69.0	62.0	60.0	64	14.5	-	1	-
H154	131	33	75	100	50	0.71	5.26	4.81	80.0	75.0	40.0	50.0	5.60	5.70	5.20	1.0	0.9	0.7	60.0	56.5	46.0	53.5	19.0	-	1	-
H155	74	33	43	100	50	2.55	5.27	3.33	70.0	60.0	43.0	58.0	5.50	5.60	5.40	1.2	0.4	0.4	65.5	59.5	53.5	58.5	19.0	-	-	1
H156	54	59	55	112	55	1.30	1.57	2.04	62.0	59.0	42.0	45.0	5.40	5.50	5.40	1.2	0.9	0.4	59.5	47.5	42.0	47	28.5	2	1	-
H157	118	28	57	100	50	0.18	3.85	0.09	70.0	65.0	36.0	25.0	5.70	5.75	5.60	2.0	2.2	2.0	60.0	57.0	51.0	56	19.0	-	1	-
H158	122	47	100	160	80	1.45	2.45	1.11	64.0	60.0	37.0	57.0	5.50	5.50	5.40	0.3	0.9	0.7	65.0	57.5	50.0	54.5	22.0	1	-	-
H159	88	59	90	160	80	1.28	5.45	4.36	54.0	50.0	46.0	56.0	5.45	5.60	5.50	0.1	0.2	0.5	63.0	54.5	48.0	53	18.0	-	-	-
H160	262	26	120	170	85	1.30	3.30	2.36	100	65.0	63.0	74.0	5.40	5.60	5.50	0.9	1.2	0.0	64.5	57.5	53.5	56.5	17.0	-	-	-
H161	189	44	146	170	85	0.18	3.85	0.09	120	90.0	60.0	50.0	5.70	5.75	5.60	2.0	2.2	2.0	66.0	58.0	52.0	58	15.0	-	-	-
H162	142	26	65	100	50	0.64	7.14	3.36	67.0	62.0	42.0	49.0	5.50	5.60	5.50	0.3	0.4	0.5	59.0	52.0	49.0	54.5	19.0	-	-	-
H163	72	44	55	130	65	2.10	7.50	2.59	90.0	80.0	22.0	33.0	5.25	5.50	5.40	0.2	0.3	0.4	64.0	65.0	50.0	50	16.5	-	-	-
H164	95	57	95	130	65	0.18	5.87	4.63	72.0	70.0	37.0	43.0	5.45	5.55	5.40	0.2	0.2	0.2	62.5	55.0	49.0	55.5	24.5	-	-	-
H165	167	27	79	135	65	2.02	5.61	0.09	58.0	50.0	44.0	60.0	5.45	5.70	5.50	0.5	0.3	0.1	54.5	50.5	46.0	49.5	11.0	-	-	-
H166	251	23	100	135	65	1.61	2.41	1.98	77.0	63.0	50.0	54.0	5.60	5.60	5.55	0.1	0.1	0.1	61.0	56.0	50.0	57	15.5	-	-	-
H167	194	25	85	230	115	2.73	7.21	5.93	66.0	56.0	38.0	41.0	5.50	5.55	5.40	0.1	0.1	0.2	60.5	55.0	46.0	51.5	17.0	-	1	-
H168	30	71	38	126	60	0.82	2.07	0.09	70.0	52.0	44.0	45.0	5.50	5.80	5.30	0.8	1.0	0.5	54.5	45.0	33.0	36	23.0	-	1	-
H169	222	12	46	126	60	2.31	2.74	0.20	75.0	73.0	51.0	65.0	5.40	5.50	5.00	1.6	1.2	1.3	64.5	58.0	52.0	58	16.5	-	-	-
H170	155	31	84	115	55	0.58	1.11	0.69	85.0	80.0	43.0	50.0	5.20	5.40	5.10	3.0	3.0	0.8	65.5	59.0	53.0	58	14.5	-	-	-
H171	210	28	101	160	75	0.18	3.85	0.09	75.0	65.0	40.0	44.0	5.70	5.75	5.60	2.0	2.2	2.0	62.0	68.0	51.0	57	19.0	1	1	-
H172	109	31	60	210	105	4.00	8.93	5.36	100	75.0	50.0	38.0	5.50	5.60	5.60	0.3	0.4	0.3	65.0	56.5	49.0	53.5	17.0	1	1	-
H173	374	20	129	210	105	1.09	1.43	0.93	100	62.0	42.0	48.0	5.50	5.60	5.40	0.1	0.6	0.8	61.5	51.0	43.5	51.5	19.0	-	1	-
H174	265	16	75	245	120	1.98	2.19	0.28	74.0	65.0	52.0	54.0	5.30	5.50	5.40	0.1	0.2	0.2	63.0	55.0	51.0	55.5	16.5	-	1	-
H175	177	31	95	245	120	3.17	4.45	3.67	120	62.0	46.0	56.0	5.20	5.50	5.45	0.7	0.4	0.3	63.0	56.0	48.5	54	21.5	-	-	-
H176	139	28	68	142	70	0.54	4.00	2.12	70.0	56.0	33.0	42.0	4.65	5.55	6.60	0.1	0.0	0.2	64.0	49.0	44.0	56	25.5	-	-	-
H177	79	25	35	102	50	0.73	3.73	0.09	62.0	53.0	35.0	37.0	5.50	5.50	5.60	0.0	0.1	0.0	59.5	52.5	45.5	51.5	17.5	-	-	-
H178	158	20	54	102	50	2.31	4.09	1.45	60.0	52.0	35.0	37.0	5.40	5.50	5.50	0.3	0.9	0.8	56.0	53.5	46.5	51	11.0	1	1	-

Table A1.2 Spread sheet prepared for spot speed measurement

Curve No: H101			Trap Length = 10m						
	Preceding Tangent Point		Start Point		Middle Point		End Point		Speed differential
Vehicle Nos	Time (Sec)	Speed(km/h)	Time (Sec)	Speed(km/h)	Time (Sec)	Speed (km/h)	Time (Sec)	Speed (km/h)	$(\Delta V)_{TMM}$ (km/h)
1	0.52	69.2	0.56	64.3	0.62	58.1	0.58	62.1	11.2
2	0.59	61.0	0.64	56.3	0.65	55.4	0.57	63.2	5.6
3	0.57	63.2	0.59	61.0	0.68	52.9	0.65	55.4	10.2
4	0.67	53.7	0.7	51.4	0.82	43.9	0.74	48.6	9.8
5	0.66	54.5	0.75	48.0	0.83	43.4	0.7	51.4	11.2
6	0.6	60.0	0.64	56.3	0.7	51.4	0.66	54.5	8.6
7	0.58	62.1	0.65	55.4	0.78	46.2	0.76	47.4	15.9
8	0.52	69.2	0.66	54.5	0.78	46.2	0.6	60.0	23.1
9	0.65	55.4	0.69	52.2	0.76	47.4	0.72	50.0	8.0
10	0.72	50.0	0.74	48.6	0.82	43.9	0.81	44.4	6.1
11	0.5	72.0	0.56	64.3	0.57	63.2	0.56	64.3	8.8
12	0.66	54.5	0.72	50.0	0.85	42.4	0.79	45.6	12.2
13	0.63	57.1	0.7	51.4	0.83	43.4	0.8	45.0	13.8
14	0.56	64.3	0.68	52.9	0.75	48.0	0.7	51.4	16.3
15	0.64	56.3	0.66	54.5	0.7	51.4	0.65	55.4	4.8
16	0.72	50.0	0.85	42.4	1.03	35.0	0.82	43.9	15.0

17	0.59	61.0	0.63	57.1	0.73	49.3	0.65	55.0	11.7
18	0.58	62.1	0.6	60.0	0.65	55.4	0.64	56.3	6.7
19	0.72	50.0	0.78	46.2	0.88	40.9	0.83	43.4	9.1
20	0.53	67.9	0.59	61.0	0.68	52.9	0.59	61.0	15.0
21	0.65	55.4	0.79	45.6	0.9	40.0	0.76	47.4	15.4
22	0.54	66.7	0.59	61.0	0.66	54.5	0.65	55.4	12.1
23	0.69	52.2	0.88	40.9	0.98	36.7	0.8	45.0	15.4
24	0.57	63.2	0.61	59.0	0.66	54.5	0.63	57.1	8.6
25	0.58	62.1	0.62	58.1	0.71	50.7	0.65	55.4	11.4
26	0.59	61.0	0.61	59.0	0.7	51.4	0.67	53.7	9.6
27	0.67	53.7	0.7	51.4	0.76	47.4	0.73	49.3	6.4
28	0.72	50.0	0.73	49.3	0.75	48.0	0.74	48.6	2.0
29	0.63	57.1	0.74	48.6	0.76	47.4	0.66	54.5	9.8
30	0.65	55.4	0.8	45.0	0.83	43.4	0.75	48.0	12.0
31	0.6	60.0	0.65	55.4	0.67	53.7	0.61	59.0	6.3
32	0.7	51.4	0.72	50.0	0.78	46.2	0.67	53.7	5.3
33	0.53	67.9	0.58	62.1	0.67	53.7	0.57	63.2	14.2
34	0.54	66.7	0.66	54.5	0.85	42.4	0.55	65.5	24.3
35	0.54	66.7	0.65	55.4	0.87	41.4	0.62	58.1	25.3
36	0.69	52.2	0.85	42.4	1.1	32.7	0.72	50.0	19.4
37	0.52	69.2	0.56	64.3	0.78	46.2	0.67	53.7	23.1
38	0.8	45.0	0.92	39.1	0.73	49.3	0.84	42.9	-4.3
39	0.55	65.5	0.63	57.1	0.84	42.9	0.8	45.0	22.6
40	0.6	60.0	0.7	51.4	0.75	48.0	0.63	57.1	12.0
41	0.58	62.1	0.61	59.0	0.69	52.2	0.62	58.1	9.9

42	0.63	57.1	0.71	50.7	0.83	43.4	0.77	46.8	13.8
43	0.5	72.0	0.65	55.4	0.78	46.2	0.58	62.1	25.8
44	0.8	45.0	0.89	40.4	1.23	29.3	0.92	39.1	15.7
45	0.62	58.1	0.64	56.3	0.76	47.4	0.63	57.1	10.7
46	0.52	69.2	0.62	58.1	0.78	46.2	0.65	55.4	23.1
47	0.65	55.4	0.67	53.7	0.73	49.3	0.7	51.4	6.1
48	0.65	55.4	0.7	51.4	0.71	50.7	0.69	52.2	4.7
49	0.55	65.5	0.74	48.6	0.85	42.4	0.66	54.5	23.1
50	0.57	63.2	0.78	46.2	0.89	40.4	0.59	61.0	22.7
51	0.52	69.2	0.55	65.5	0.69	52.2	0.64	56.3	17.1
52	0.78	46.2	0.81	44.4	0.91	39.6	0.86	41.9	6.6
53	0.67	53.7	0.74	48.6	0.76	47.4	0.74	48.6	6.4
54	0.55	65.5	0.57	63.2	0.62	58.1	0.78	46.2	7.4
55	0.54	66.7	0.58	62.1	0.69	52.2	0.6	60.0	14.5
56	0.56	64.3	0.6	60.0	0.65	55.4	0.76	47.4	8.9
57	0.53	67.9	0.68	52.9	0.81	44.4	0.7	51.4	23.5
58	0.56	64.3	0.7	51.4	0.73	49.3	0.6	60.0	15.0
59	0.67	53.7	0.69	52.2	0.9	40.0	0.71	50.7	13.7
60	0.71	50.7	0.72	50.0	0.9	40.0	0.89	40.4	10.7
61	0.8	45.0	1.03	35.0	1.09	33.0	0.9	40.0	12.0
62	0.63	57.1	0.82	43.9	1.12	32.1	0.72	50.0	25.0
63	0.62	58.1	0.74	48.6	0.96	37.5	0.93	38.7	20.6
64	0.68	52.9	0.71	50.7	0.82	43.9	0.8	45.0	9.0
65	0.56	64.3	0.63	57.1	0.68	52.9	0.65	55.4	11.3
66	0.52	69.2	0.62	58.1	0.72	50.0	0.69	52.2	19.2

67	0.63	57.1	0.78	46.2	1.01	35.6	0.8	45.0	21.5
68	0.66	54.5	0.71	50.7	0.74	48.6	0.72	50.0	5.9
69	0.62	58.1	0.81	44.4	1.07	33.6	0.69	52.2	24.4
70	0.61	59.0	0.67	53.7	0.75	48.0	0.66	54.5	11.0
71	0.56	64.3	0.65	55.4	0.83	43.4	0.59	61.0	20.9
72	0.54	66.7	0.55	65.5	0.82	43.9	0.7	51.4	22.8
73	0.57	63.2	0.71	50.7	0.78	46.2	0.66	54.5	17.0
74	0.62	58.1	0.8	45.0	0.9	40.0	0.78	46.2	18.1
75	0.62	58.1	0.8	45.0	0.85	42.4	0.72	50.0	15.7
76	0.6	60.0	0.61	59.0	0.68	52.9	0.7	51.4	7.1
77	0.51	70.6	0.72	50.0	0.79	45.6	0.72	50.0	25.0
78	0.7	51.4	0.76	47.4	0.99	36.4	0.92	39.1	15.1
79	0.61	59.0	0.65	55.4	0.68	52.9	0.66	54.5	6.1
80	0.7	51.4	0.72	50.0	0.87	41.4	0.79	45.6	10.0
81	0.65	55.4	0.75	48.0	0.98	36.7	0.78	46.2	18.6
82	0.66	54.5	0.68	52.9	0.99	36.4	0.7	51.4	18.2
83	0.62	58.1	0.7	51.4	0.83	43.4	0.69	52.2	14.7
84	0.72	50.0	0.75	48.0	1.1	32.7	0.92	39.1	17.3
85	0.51	70.6	0.63	57.1	0.7	51.4	0.58	62.1	19.2
86	0.58	62.1	0.62	58.1	0.78	46.2	0.62	58.1	15.9
87	0.65	55.4	0.68	52.9	0.9	40.0	0.87	41.4	15.4
88	0.74	48.6	0.84	42.9	0.88	40.9	0.87	41.4	7.7
89	0.71	50.7	0.82	43.9	0.89	40.4	0.88	40.9	10.3
90	0.51	70.6	0.65	55.4	0.93	38.7	0.71	50.7	31.9
91	0.72	50.0	0.65	55.4	0.86	41.9	0.83	43.4	8.1

92	0.65	55.4	0.78	46.2	0.9	40.0	0.81	44.4	15.4
93	0.6	60.0	0.65	55.4	0.69	52.2	0.62	58.1	7.8
94	0.6	60.0	0.77	46.8	0.78	46.2	0.69	52.2	13.8
95	0.62	58.1	0.71	50.7	0.87	41.4	0.78	46.2	16.7
96	0.5	72.0	0.65	55.4	0.84	42.9	0.71	50.7	29.1
97	0.49	73.5	0.62	58.1	0.86	41.9	0.63	57.1	31.6
98	0.61	59.0	0.65	55.4	0.77	46.8	0.7	51.4	12.3
99	0.62	58.1	0.78	46.2	0.8	45.0	0.74	48.6	13.1
100	0.59	61.0	0.68	52.9	0.78	46.2	0.6	60.0	14.9
101	0.6	60.0	0.65	55.4	0.75	48.0	0.69	52.2	12.0
102	0.54	66.7	0.63	57.1	0.83	43.4	0.68	52.9	23.3
103	0.72	50.0	0.85	42.4	1.02	35.3	0.92	39.1	14.7
104	0.65	55.4	0.86	41.9	0.88	40.9	0.7	51.4	14.5
105	0.53	67.9	0.65	55.4	0.69	52.2	0.65	55.4	15.8

Table A1.3 Frequency distribution of spot speed data

At tangent point of curve H101			
Speed range km/h	Frequency	Relative frequency %	Cumulative frequency %
0	0	0	0
4	0	0	0
8	0	0	0
12	0	0	0
24	0	0	0
28	0	0	0
32	0	0	0
36	0	0	0
40	0	0	0
44	0	0	0
48	4	3.80952	3.80952
52	13	12.381	16.1905
56	20	19.0476	35.2381
60	24	22.8571	58.1
64	13	12.381	70.4762
68	18	17.1429	87.619
72	12	11.4286	99.0476
76	1	0.95238	100
80	0	0	100
84	0	0	100
88	0	0	100
92	0	0	100
96	0	0	100
100	0	0	100
Total:	**105**	**100**	

Table A1.4 Frequency distribution of spot speed differential data

Speed differential between tangent at middle of curve H101			
Speed range km/h	Frequency	Relative frequency %	Cumulative frequency %
0	1	0.95238	0.95238
3	1	0.95238	1.90476
6	5	4.7619	6.66667
9	18	17.1429	23.8095
12	20	19.0476	42.8571
15	17	16.1905	59.0476
18	16	15.2381	74.2857
21	8	7.61905	81.9048
24	10	9.52381	91.4286
27	6	5.71429	97.1429
30	1	0.95238	98.0952
33	2	1.90476	100
36	0	0	100
39	0	0	100
42	0	0	100
45	0	0	100.0
48	0	0	100
51	0	0	100
54	0	0	100
57	0	0	100
60	0	0	100
Total:	**105**	**100**	

APPENDIX 2

CORRELATION MATRICES

Table A2.1 Correlation matrix for the development of operating speed models of Class A curves

	R_c	$1/R_c$	Δ	L_H	PTL	V_S	V_M	V_E	e_S	e_M	e_E	SD_{TS}	SD_{TM}	SD_S	SD_M	$\sqrt{R_c}$	R_c^2	$\ln(1/R_c)$
R_c	1																	
$1/R_c$	-0.77	1																
Δ	-0.64	0.76	1															
L_H	0.65	-0.56	-0.03	1														
PTL	0.32	-0.25	0.05	0.56	1													
V_S	.605(**)	-.606(**)	-.400(*)	.535(**)	0.11	1												
V_M	.610(**)	-.609(**)	-.399(*)	.556(**)	0.082	.884(**)	1											
V_E	.600(**)	-.690(**)	-.477(**)	.563(**)	0.121	.851(**)	.828(**)	1										
e_S	-0.06	0.14	0.08	-0.03	0.03	-0.09	0.01	-0.03	1									
e_M	-0.45	0.42	0.40	-0.20	-0.32	-0.24	-0.12	-0.13	0.203	1								
e_E	-0.18	-0.04	-0.01	0.01	0.19	-0.17	-0.08	-0.01	-0.01	0.26	1							
SD_{TS}	0.42	-0.34	0.03	0.70	0.75	.508(**)	.484(**)	.483(**)	0.02	-0.12	0.07	1						
SD_{TM}	0.21	-0.13	0.15	0.47	0.65	0.21	0.23	0.23	-0.12	-0.12	0.10	0.77	1					
SD_S	0.00	-0.15	0.07	0.21	0.12	0.26	.407(*)	0.28	-0.11	0.41	0.17	0.35	0.24	1				
SD_M	0.24	-0.32	-0.10	0.42	0.37	0.48	.454(**)	.551(**)	-0.03	0.19	0.09	0.70	0.50	0.57	1			
$\sqrt{R_c}$	0.99	-0.85	-0.69	0.66	0.31	.644(**)	.638(**)	.656(**)	-0.06	-0.45	-0.14	0.42	0.19	0.03	0.29	1		
R_c^2	0.97	-0.62	-0.54	0.62	0.33	.505(**)	.536(**)	.477(**)	-0.07	-0.43	-0.20	0.39	0.24	-0.05	0.16	0.92	1	
$\ln(1/R_c)$	-0.95	0.92	0.74	-0.65	-0.30	-.663(**)	-.652(**)	-.697(**)	0.08	0.45	0.09	-0.42	-0.17	-0.07	-0.32	-0.99	-0.85	1

Where,
.** Correlation is significant at the 0.01 level (2-tailed) and * Correlation is significant at the 0.05level (2-tailed).

Table A2.2 Correlation matrix developed for Class B curves

	R_c	$1/R_c$	Δ	L_H	PTL	V_S	V_M	V_E	e_S	e_M	e_E	SD_{TS}	SD_{TM}	SD_S	SD_M	$\sqrt{R_c}$	R_c^2	$\ln(1/R_c)$
R_c	1																	
$1/R_c$	-0.86	1																
Δ	-0.69	0.82	1															
L_H	0.67	-0.63	-0.28	1														
PTL	0.44	-0.22	-0.03	0.62	1													
V_S	.598(**)	-.520(**)	-.402(*)	0.37	0.341	1												
V_M	.628(**)	-.629(**)	-.444(*)	.512(**)	0.26	.784(**)	1											
V_E	.621(**)	-.651(**)	-.443(*)	.424(*)	0.267	.868(**)	.904(**)	1										
e_S	0.26	-0.30	-0.30	0.09	0.00	0.06	0.04	0.15	1									
e_M	0.06	-0.13	-0.14	-0.06	0.01	0.05	0.19	0.14	0.52	1								
e_E	-0.09	-0.05	-0.06	0.07	-0.05	-0.14	0.07	0.05	0.16	0.43	1							
SD_{TS}	0.65	-0.50	-0.31	0.62	0.48	.816(**)	.621(**)	.667(**)	-0.06	-0.12	-0.23	1						
SD_{TM}	0.61	-0.44	-0.34	0.47	0.57	.608(**)	0.37	.399(*)	0.06	-0.08	-0.33	0.71	1					
SD_S	0.23	-0.38	-0.29	0.18	0.27	.459(*)	.466(*)	.524(**)	-0.09	0.16	-0.10	0.40	0.50	1				
SD_M	0.37	-0.49	-0.47	0.23	0.18	.558(**)	.647(**)	.630(**)	-0.06	0.06	0.03	0.37	0.44	0.83	1			
$\sqrt{R_c}$	0.99	-0.92	-0.75	0.68	0.38	.593(**)	.645(**)	.641(**)	0.26	0.06	-0.07	0.63	0.58	0.27	0.42	1		
R_c^2	0.96	-0.71	-0.56	0.62	0.56	.580(**)	.573(**)	.570(**)	0.28	0.09	-0.09	0.64	0.63	0.15	0.27	0.91	1	
$\ln(1/R_c)$	-0.96	0.97	0.79	-0.68	-0.32	-.578(**)	-.650(**)	-.653(**)	-0.27	-0.08	0.03	-0.60	-0.54	-0.32	-0.46	-0.99	-0.85	1

Where

.** Correlation is significant at the 0.01 level (2-tailed) and * Correlation is significant at the 0.05level (2-tailed).

Table A2.3 Correlation matrix for the development of operating speed models of Class C curves

	R_c	$1/R_c$	Δ	L_H	PTL	V_S	V_M	V_E	e_S	e_M	e_E	SD_{TS}	SD_{TM}	SD_S	SD_M	$1/R_c^2$	$1/\sqrt{R_c}$	$\ln(1/R_c)$
R_c	1																	
$1/R_c$	-0.76	1																
Δ	-0.64	0.68	1															
L_H	0.36	-0.42	0.30	1														
PTL	0.19	-0.01	-0.17	-0.18	1													
V_S	.497(*)	-.598(**)	-0.36	0.36	0.35	1												
V_M	.668(**)	-.690(**)	-.444(*)	.471(*)	0.363	732(**)	1											
V_E	0.23	-0.21	-0.31	0.02	0.30	640(**)	0.40	1										
e_S	0.13	-0.23	-0.36	-0.26	-0.08	0.23	0.16	0.28	1									
e_M	-0.51	0.55	0.73	0.16	0.13	-0.141	-0.26	-0.35	-0.47	1								
e_E	-0.31	0.23	0.44	0.27	-0.21	-0.273	-0.30	-0.01	-0.06	0.26	1							
SD_{TS}	0.41	-0.18	-0.06	0.25	0.49	0.432	0.36	0.19	-0.17	0.03	-0.46	1						
SD_{TM}	0.52	-0.29	-0.01	0.40	0.51	502(*)	0.40	0.19	-0.17	0.02	-0.32	0.88	1					
SD_S	0.47	-0.50	-0.17	0.46	0.43	496(*)	660(**)	0.23	-0.09	0.01	-0.39	0.55	0.54	1				
SD_M	0.27	-0.38	-0.21	0.14	0.45	0.337	0.334	0.09	-0.05	0.05	-0.34	0.26	0.30	0.72	1			
$1/R_c^2$	-0.59	0.96	0.62	-0.38	0.02	-.562(**)	-.606(**)	-0.13	-0.17	0.47	0.18	-0.06	-0.17	-0.43	-0.39	1		
$1/\sqrt{R_c}$	-0.85	0.99	0.70	-0.43	-0.04	-.598(**)	-.718(**)	-0.24	-0.23	0.58	0.26	-0.25	-0.36	-0.52	-0.36	0.90	1	
$\ln(1/R_c)$	0.93	-0.94	-0.70	0.41	0.09	580(**)	726(**)	0.25	0.22	-0.58	-0.28	0.32	0.42	0.52	0.33	-0.82	-0.98	1

Where,
.** Correlation is significant at the 0.01 level (2-tailed) and * Correlation is significant at the 0.05level (2-tailed).

Table A2.4. Correlation matrix developed for the development of speed differential models of Class B curves

	R_c	$1/R_c$	Δ	L_H	PTL	ΔV_{85TMM}	PTL/R_c	R_c/PTL	SD_{TS}	SD_{TM}	SD_S	SD_M	$\sqrt{R_c}$	R_c^2	$1/\sqrt{R_c}$	$1/R_c^2$	$PTL/\sqrt{R_c}$
R_c	1																
$1/R_c$	-0.90	1															
Δ	-0.79	0.82	1														
L_H	0.51	-0.50	-0.19	1													
PTL	-0.17	0.14	0.27	0.50	1												
$(\Delta V)_{85TMM}$	-.668(**)	.644(**)	.534(*)	-0.158	.521(*)	1											
PTL/R_c	-0.81	0.91	0.83	-0.25	0.50	.762(**)	1										
R_c/PTL	0.88	-0.77	-0.73	0.19	-0.59	-.742(**)	-0.85	1									
SD_{TS}	0.13	-0.08	-0.03	0.29	0.39	0.15	0.09	-0.10	1								
SD_{TM}	-0.02	0.04	-0.06	0.17	0.41	0.27	0.20	-0.23	0.67	1							
SD_S	0.29	-0.38	-0.28	0.15	0.17	-0.322	-0.27	0.11	0.31	0.40	1						
SD_M	0.47	-0.51	-0.48	0.14	0.00	-.475(*)	-0.44	0.32	0.56	0.48	0.67	1					
$\sqrt{R_c}$	0.99	-0.94	-0.82	0.52	-0.16	-.674(**)	-0.85	0.87	0.12	-0.03	0.32	0.49	1				
R_c^2	0.98	-0.81	-0.72	0.49	-0.18	-.640(**)	-0.73	0.87	0.15	-0.03	0.24	0.42	0.96	1			
$1/\sqrt{R_c}$	-0.94	0.99	0.84	-0.51	0.14	.662(**)	0.90	-0.81	-0.09	0.03	-0.37	-0.51	-0.97	-0.87	1		
$1/R_c^2$	-0.80	0.98	0.78	-0.46	0.13	.594(**)	0.91	-0.68	-0.05	0.06	-0.38	-0.48	-0.85	-0.68	0.95	1	
$PTL/\sqrt{R_c}$	-0.71	0.75	0.75	-0.02	0.75	.786(**)	0.94	-0.89	0.21	0.29	-0.14	-0.35	-0.73	-0.66	0.75	0.73	1

Where,
.** Correlation is significant at the 0.01 level (2-tailed) and * Correlation is significant at the 0.05level (2-tailed).

Table A2.5 Correlation matrix developed for the development of speed differential models of Class C curves

	R_c	$1/R_c$	Δ	L_H	PTL	ΔV_{85TMM}	PTL/ R_c	R_c/ PTL	es	SD_{TS}	SD_{TM}	SD_S	SD_M	$\sqrt{R_c}$	R_c^2	$1/\sqrt{R_c}$	$1/R_c^2$	PTL/$\sqrt{R_c}$
R_c	1																	
$1/R_c$	-0.86	1																
Δ	-0.64	0.67	1															
L_H	0.16	-0.23	0.48	1														
PTL	-0.34	0.29	0.16	-0.15	1													
$(\Delta V)_{85TMM}$	-.577(**)	0.42	.530(*)	0.19	0.14	1												
PTL/ R_c	-0.72	0.81	0.51	-0.25	0.78	0.33	1											
R_c /PTL	0.89	-0.76	-0.57	0.15	-0.64	-.522(*)	-0.82	1										
es	0.44	-0.27	-0.44	-0.27	0.02	-.450(*)	-0.16	0.18	1									
SD_{TS}	-0.12	0.19	0.25	0.19	0.38	0.384	0.38	-0.20	-0.16	1								
SD_{TM}	-0.17	0.14	0.36	0.30	0.32	.522(*)	0.31	-0.24	-0.19	0.82	1							
SD_S	0.12	-0.25	0.06	0.25	0.28	-0.022	0.00	0.03	-0.11	0.34	0.19	1						
SD_M	0.02	-0.26	-0.11	0.01	0.28	0.164	-0.03	-0.11	-0.06	0.11	0.13	0.69	1					
$\sqrt{R_c}$	0.99	-0.92	-0.66	0.19	-0.34	-.552(*)	-0.76	0.88	0.41	-0.14	-0.16	0.16	0.08	1				
R_c^2	0.98	-0.75	-0.59	0.10	-0.32	-.597(**)	-0.63	0.86	0.45	-0.09	-0.17	0.05	-0.08	0.95	1			
$1/\sqrt{R_c}$	-0.92	0.99	0.68	-0.22	0.32	.470(*)	0.81	-0.82	-0.32	0.17	0.15	-0.23	-0.21	-0.96	-0.83	1		
$1/R_c^2$	-0.72	0.97	0.63	-0.21	0.23	0.31	0.77	-0.62	-0.17	0.22	0.14	-0.25	-0.32	-0.79	-0.58	0.92	1	
PTL/$\sqrt{R_c}$	-0.61	0.62	0.38	-0.22	0.93	0.285	0.96	-0.80	-0.12	0.39	0.33	0.12	0.12	-0.64	-0.55	0.64	0.56	1

Where.

** Correlation is significant at the 0.01 level (2-tailed) and * Correlation is significant at the 0.05level (2-tailed).

APPENDIX 3

ALIGNMENT INDICES FOR INDIVIDUAL FEATURES

Table A3.1 Data collected and alignment indices for individual features

Section	Notation	R_c(m)	Δ (deg)	D (deg)	L_H(m)	PTL(m)	DTL(m)	G1%	G2%	A %	Number of accidents			Alignment indices	
											FA	GI	SI	R/AR	TL/ATL
	H30	291	32	5.9	163.0	105.0	360	-	-		-	-	-	0.8	0.5
13	H31	121	81	14.2	172.0	360.0	255	-	-		1	1	2	0.3	1.6
	H32	635	15	2.7	170.0	255.0	175	-	-		1	1	1	1.8	1.1
						175.0									0.8
	H33	45	104	38.2	82.0	350.0	120	-	-		-	1	1	0.2	2.3
	CC10	264	14	6.5	66.0	120.0	110	3.6	-3.3	6.9	-	-	-	1.3	0.8
	V13	-			240.0	110.0	83	3.5	-2.7	6.2	-	-	1		0.7
	V14	-			237.0	83.0	165	4.9	-2.8	7.7	-	1	-		0.5
14	H34	297	51	5.8	263.0	165.0	84	-	-		-	-	-	1.5	1.1
						84.0									0.6
	V15	-			378.0	240.0	270	4.1	-5.4	9.5	-	1	1		1.9
15	CC11	427	47	4.0	350.0	270.0	120	2.9	-0.6	3.5	-	-	1	1.6	2.1
	V16	-			270.0	120.0	210	3.5	-4.6	8.1	-	-	-		0.9
	V17	-			435.0	210.0	250	2.8	-5.2	8.0	-	-	-		1.6
	AH15	85	74	20.2	110.0	250.0	0	-	-		-	-	-	0.3	2.0
	AH16	44	159	39.1	122.0	0.0	0	-	-		-	-	-	0.2	0.0

	AH17	123	84	14.0	180.0	0.0	90	-	-		-	-	-	0.5	0.0
	V18	-			260.0	90.0	120	3.4	-2.8	6.2	-	1	-		0.7
	H35	865	7	2.0	105.0	120.0	0	-	-		-	1	-	3.3	0.9
	H36	47	110	36.6	90.0	0.0	100	-	-		1	-	1	0.2	0.0
						100.0									0.8
	CC12	74.2	79	23.5	102.0	30.0	125	-3.0	1.2	4.2	-	-	-	0.4	0.4
	H37	426	35	4.1	259.0	125.0	152	-	-		-	-	-	2.0	1.7
	H38	188	23	9.3	75.0	152.0	100	-	-		-	-	-	0.9	2.1
	V19	-			115.0	100.0	56	4.6	-1.3	5.9	-	-	-		1.4
	H39	170.7	37	10.2	110.0	56.0	126	-	-		-	-	-	0.8	0.8
16	CC13	183.6	35	9.5	113.0	126.0	0	0.6	2.9	2.3	-	1	1	0.9	1.7
	CC14	266	30	6.6	137.0	0.0	100	2.4	1.8	0.6	1	1	-	1.3	0.0
	V20	-			102.0	100.0	55	3.8	-3.2	7.0	-	-	-		1.4
	AH18	274	23	6.4	109.0	55.0	0	-	-		-	1	1	1.3	0.7
	AH19	94	100	18.6	164.0	0.0	70	-	-		-	-	-	0.4	0.0
						70.0									0.9
	V21	-			182.0	40.0	140	3.5	-2.1	5.6	-	-	-		0.4
17	CC15	108.6	61	16.1	115.0	140.0	0	2.3	0.5	1.8	-	-	1	0.6	1.3
	CC16	117	42	14.9	86.0	0.0	100	2.9	1.0	1.9	1	-	-	0.7	0.0
	CC17	112	45	15.6	88.0	100.0	258	3.0	0.5	2.5	-	-	-	0.7	1.0
	CC18	350	25	5.0	154.0	258.0	87	-1.5	-3.3	1.8	-	-	-	2.0	2.5
						87.0									0.8
	CC19	53.5	90	32.6	84.0	57.0	42	2.6	1.4	1.2	-	1	-	0.3	0.6
	CC20	185	34	9.4	110.0	42.0	125	3.8	2.6	1.2	-	-	-	1.2	0.4
	CC21	271	19	6.4	89.0	125.0	0	4.5	0.0	4.5	-	-	-	1.7	1.3

18	CC22	98	33	17.8	57.0	0.0	25	5.0	0.5	4.5	-	-	-	0.6	0.0
	CC23	141	28	12.4	69.0	25.0	190	5.7	2.0	3.7	-	-	-	0.9	0.3
	H40	305	30	5.7	161.0	190.0	264	-	-		-	-	-	1.9	2.0
	H41	125	56	14.0	122.0	264.0	104	-	-		-	1	-	0.8	2.8
	H42	88	79	19.8	121.0	104.0	40	-	-		1	1	2	0.6	1.1
						40.0									0.4
	V22	-			100.0	100.0	59	3.7	-2.8	6.5	-	-	-		1.3
	CC24	142	50	12.3	125.0	59.0	83	-2.5	-2.1	0.4	-	1	-	1.0	0.8
	CC25	88.5	45	19.7	70.0	83.0	0	-2.3	-2.2	0.1	-	-	-	0.6	1.1
	CC26	65	85	26.9	96.5	0.0	25	-2.2	0.5	2.7	-	-	-	0.5	0.0
	H43	106	32	16.5	60.0	25.0	170	-	-		-	1	-	0.8	0.3
	V23	-			150.0	170.0	58	4.2	-3.4	7.6	-	2	-		2.2
19	AH20	230	22	7.6	90.0	58.0	64	-	-		-	-	-	1.7	0.8
	CC27	209	36	8.4	130.0	64.0	105	2.1	0.2	1.9	-	-	-	1.5	0.8
	CC28	193	34	9.0	115.0	105.0	105	-1.8	-3.2	1.4		-	-	1.4	1.4
	H44	88	64	19.8	98.5	105.0	0	-	-		1	1	-	0.6	1.4
	H45	128	25	13.6	56.0	0.0	140	-	-		-	1	1	0.9	0.0
	V24	-			220.0	140.0	80	5.5	-3.0	8.5	-	2	-		1.8
						80.0									1.1
	H46	254	41	6.9	180.0	300.0	0	-	-		-	-	-	1.4	3.6
	AH21	220	33	7.9	128.0	0.0	0	-	-		-	-	-	1.3	0.0
	H47	114	69	15.3	137.0	0.0	120	-	-		-	-	-	0.6	0.0
	V25	-			215.0	120.0	27	3.7	-3.3	7.0	-	-	-		1.4

20	CC29	130	40	13.4	91.0	27.0	0	-2.4	0.0	2.4	-	-	-	0.7	0.3
	H48	153	36	11.4	96.0	0.0	175				-	1	-	0.9	0.0
	V26	-			165.0	175.0	100	4.5	-2.0	6.5	-	-	-		2.1
	H49	237	18	7.4	75.0	100.0	0	-	-		-	-	1	1.4	1.2
	H50	120	34	14.6	71.0	0.0	120	-	-		-	-	-	0.7	0.0
						120.0									1.4
	H51	57.9	49	30.2	50.0	40.0	175	-	-		-	-	1	0.5	0.3
	H52	208.6	18	8.4	65.0	175.0	45	-	-		-	-	-	1.7	1.2
	H53	87.5	56	20.0	86.0	45.0	423	-	-		-	-	-	0.7	0.3
21	H54	125.8	32	13.9	71.0	423.0	45	-	-		-	-	1	1.0	2.9
						45.0									0.3
	V27	-			140.0	120.0	50	5.5	-1.5	7.0	-	-	-		1.6
	AH22	155.8	22	11.2	59.8	50.0	0	-	-		-	-	-	1.1	0.7
	AH23	67.2	41	26.0	48.0	0.0	100	-	-		-	-	-	0.5	0.0
	V28	-			240.0	100.0	60	3.7	-3.7	7.4	-	-	-		1.3
	CC30	390.7	11	4.5	75.1	60.0	115	-2.8	-2.8	0.0	-	-	-	2.8	0.8
	CC31	100.3	53	17.4	93.2	115.0	84	-3.9	-3.9	0.0	-	-	-	0.7	1.5
	AH24	26.8	81	65.2	38.1	84.0	0	-	-		-	-	-	0.2	1.1
	AH25	33.6	81	52.0	47.7	0.0	230	-	-		-	-	-	0.2	0.0
	CC32	168	42	10.4	122.6	230.0	40	0.5	-3.8	4.3	-	-	-	1.2	3.1
	H55	69.7	48	25.1	59.0	40.0	144	-	-		-	-	-	0.5	0.5
	H56	180.1	30	9.7	94.0	144.0	0	-	-		-	-	-	1.3	1.9
22	CC33	189.6	21	9.2	69.5	0.0	30	2.9	-1.2	4.1	-	-	-	1.4	0.0
						30.0									0.4
23	H57	179.5	22	9.7	70.0	180.0	185	-	-		-	-	-	1.0	1.3

	H58	105.8	33	16.5	61.0	185.0	104	-	-		-	-	-	0.6	1.4
	CC34	412.8	6	4.2	45.0	104.0	80	3.2	3.2	0.0	-	-	-	2.3	0.8
	H59	103.7	36	16.8	65.0	80.0	180	-	-		-	-	-	0.6	0.6
	H60	77.8	37	22.4	50.0	180.0	80	-	-		-	-	-	0.4	1.3
						80.0									0.6
	CC35	44	91	39.7	70.0	45.0	105	-4.0	-4.0	0.0	-	1	-	0.1	0.4
	H61	41.9	70	41.7	51.0	105.0	169	-	-		-	-	-	0.1	1.0
	H62	259	22	6.7	100.0	169.0	130	-	-		-	-	-	0.8	1.6
	V29	-			200.0	130.0	70	2.9	-3.2	6.1	-	-	-		1.2
	AH26	550.3	17	3.2	168.0	70.0	258	-	-		-	-	-	1.7	0.6
	H63	185.1	40	9.4	130.0	258.0	0	-	-		-	-	-	0.6	2.4
	CC36	834.6	12	2.1	180.0	0.0	145	2.8	2.8	0.0	-	-	-	2.6	0.0
	V30	-			130.0	145.0	70	3.8	-1.5	5.3	-	-	-		1.3
24	AH27	319.5	17	5.5	93.0	70.0	86	-	-		-	-	-	1.0	0.6
						86.0									0.8
	H64	217.1	45	8.0	169.0	0.0	244	-	-		-	-	-	1.0	0.0
	H65	156.6	33	11.2	90.0	244.0	80	-	-		-	-	-	0.7	2.6
	CC37	341.6	8	5.1	45.0	80.0	150	3.1	3.1	0.0	-	-	-	1.6	0.8
	H66	261.7	17	6.7	78.0	150.0	75.2	-	-		-	-	-	1.2	1.6
25	CC38	112.1	25	15.6	48.8	75.2	20	-2.9	-2.9	0.0	-	-	-	0.5	0.8
						20.0									0.2
26	H67	84.1	59	20.8	86.0	127.0	141	-	-		-	-	1	0.4	1.4
	CC39	235.6	23	7.4	95.0	141.0	0	3.5	3.5	0.0	-	-	-	1.2	1.5
	H68	324.3	18	5.4	100.0	0.0	125	-	-		-	1	2	1.6	0.0
	AH28	169.8	44	10.3	130.0	125.0	60	-	-		-	-	-	0.9	1.4

	AH29	279.6	22	6.2	105.0	60.0	25	-	-		-	-	-	1.4	0.7
	H69	162.6	30	10.7	85.0	25.0	155	-	-		-	2	2	0.8	0.3
	H70	122.8	54	14.2	116.0	155.0	86	-	-		-	-	-	0.6	1.7
						100.0									1.1
	H71	101.5	34	17.2	61.0	35.0	133	-	-		-	1	-	0.4	0.5
	H72	336.9	18	5.2	103.0	133.0	129	-	-		-	-	-	1.3	1.8
	AH30	230.8	35	7.6	140.0	129.0	30	-	-		-	-	-	0.9	1.7
	AH31	114	40	15.3	79.0	30.0	0	-	-		-	-	-	0.4	0.4
	CC40	145.7	50	12.0	126.0	0.0	0	-4.2	-4.2	0.0	-	1	-	0.6	0.0
	CC41	550	14	3.2	131.0	0.0	218	-4.6	-4.6	0.0	-	-	-	2.1	0.0
	H73	207.8	25	8.4	90.0	218.0	175	-	-		-	-	-	0.8	2.9
	H74	313	15	5.6	80.0	175.0	65	-	-		-	-	-	1.2	2.3
	CC42	391	10	4.5	70.0	65.0	0	3.2	3.2	0.0	-	-	-	1.5	0.9
27	CC43	239.8	22	7.3	90.0	0.0	40	-2.6	-2.6	0.0	-	-	-	0.9	0.0
						45.0									0.6
	H75	117.4	46	14.9	94.0	104.0	65	-	-		-	1	-	1.1	1.6
28	CC44	98.3	61	17.8	105.0	65.0	32	2.9	2.9	0.0	-	2	-	0.9	1.0
						32.0									0.5
	H76	249.9	18	7.0	79.0	108.0	120	-	-		-	-	-	1.7	1.3
	H77	170.2	34	10.3	100.0	120.0	0	-	-		-	1	-	1.2	1.5
	H78	80.5	37	21.7	52.0	0.0	118	-	-		-	-	-	0.6	0.0
29	H79	78.8	47	22.2	64.0	118.0	59	-	-		-	-	-	0.5	1.5
						59.0									0.7
	H80	108.8	35	16.1	66.0	100.0	125	-	-		-	2	-	0.8	0.9
30	H81	255.5	13	6.8	60.0	125.0	175	-	-		-	-	1	1.9	1.1

	H82	44.9	103	38.9	81.0	175.0	43	-	-		-	1	-	0.3	1.6
						43.0									0.4
	H83	144	32	12.1	81.0	34.0	146	-	-		1	-	-	1.0	0.4
	H84	392.3	15	4.5	100.0	146.0	30	-	-		-	-	-	2.7	1.9
	H85	119.3	42	14.6	87.0	30.0	100	-	-		-	-	-	0.8	0.4
	V31	-			160.0	100.0	20	2.8	-3.0	5.8	-	-	-		1.3
	CC45	194.7	11	9.0	38.0	20.0	50	4.5	4.5	0.0	-	-	-	1.3	0.3
	H86	273	17	6.4	80.0	50.0	185	-	-		-	-	-	1.9	0.6
	CC46	89.6	48	19.5	75.0	185.0	28	4.6	4.6	0.0	-	1	-	0.6	2.4
	CC47	63	81	27.7	89.0	28.0	69	4.6	4.6	0.0	-	-	1	0.4	0.4
	AH32	112.9	36	15.5	70.0	69.0	195	-	-		-	-	-	0.8	0.9
	CC48	117.3	25	14.9	51.0	195.0	67	-2.6	-2.6	0.0	-	-	-	0.8	2.5
	AH33	124.2	34	14.1	74.0	67.0	0	-	-		-	-	-	0.9	0.9
	AH34	151.8	32	11.5	85.0	0.0	60	-	-		-	-	-	1.0	0.0
	H87	112.3	30	15.6	58.0	60.0	120	-	-		-	-	-	0.8	0.8
	H88	89.8	38	19.4	60.0	120.0	100	-	-		-	1	-	0.6	1.6
31	CC49	60.1	57	29.1	60.0	100.0	28	3.3	3.3	0.0	-	-	-	0.4	1.3
						28.0									0.4
32	H89	279.1	25	6.3	120.0	170.0	110	-	-		-	-	-	1.0	1.2
						110.0									0.8
33	H90	263.1	17	6.6	80.0	140.0	30	-	-		-	1	-	1.2	1.7
	CC50	138.2	29	12.6	70.0	30.0	50	3.4	3.4	0.0	-	-	-	0.7	0.4
	CC51	232.6	22	7.5	90.0	50.0	60	-4.1	-4.1	0.0	-	1	-	1.1	0.6
	H91	141.1	37	12.4	90.0	60.0	170	-	-		-	-	-	0.7	0.7

	H92	383.8	13	4.6	90.0	170.0	110	-	-		-	-	-	1.8	2.0
	V32	-			240.0	110.0	25	3.4	-2.4	5.8	-	-	-		1.3
	CC52	108.1	34	16.2	65.0	25.0	90	-2.3	-2.3	0.0	-	-	-	0.5	0.3
						90.0									1.1
	H93	301	23	5.8	122.0	118.0	405	-	-		-	-	-	1.3	0.8
	H94	503	17	3.5	150.0	405.0	120	-	-		-	-	-	2.1	2.7
34	V33	-			120.0	120.0	30	3.5	-3.0	6.5	-	-	-		0.8
	CC53	68.4	67	25.5	80.0	30.0	185	-4.1	-4.1	0.0	-	-	-	0.3	0.2
	H95	68.1	55	25.6	65.0	185.0	55	-	-		1	2	-	0.3	1.2
						55.0									0.4
	V34	-			240.0	110.0	50	5.1	-2.5	7.6	-	1	-		1.2
35	H96	81.6	32	21.4	45.0	100.0	55	-	-		-	-	-	1.0	1.1
						55.0									0.6
	H97	294.2	20	5.9	105.0	60.0	198	-	-		-	-	-	1.4	0.6
36	H98	125	43	14.0	93.0	198.0	44	-	-		-	-	-	0.6	2.0
						44.0									0.4
	H99	118.7	34	14.7	70.0	170.0	228	-	-		-	3	-	0.7	1.3
37.0	H100	114.5	57	15.3	113.0	228.0	106	-	-		-	-	1	0.6	1.7
	CC54	71	109	24.6	135.5	106.0	42.5	2.7	2.7	0.0	-	1	-	0.4	0.8
	CC55	55.9	67	31.2	65.0	42.5	30	-2.1	-2.1	0.0	-	-	-	0.3	0.3
	V35	-			335.0	30.0	215	4.4	-3.7	8.1	-	-	1		0.2
	H101	210	41	8.3	150.0	215.0	100	-	-		-	-	-	1.2	1.6
	H102	342	28	5.1	170.0	100.0	210	-	-		-	1	2	1.9	0.7
	H103	210	26	8.3	95.0	210.0	308	-	-		-	-	-	1.2	1.5

	H104	330	21	5.3	120.0	308.0	80	-	-		-	-	-	1.9	2.3
	CC56	204	22	8.6	80.0	80.0	130	2.6	2.6	0.0	-	-	-	1.1	0.6
	H105	370	19	4.7	125.0	225.0	70	-	-		-	1	-	2.1	1.7
	CC57	175	28	10.0	85.0	70.0	30	-2.2	-2.2	0.0	-	-	-	1.0	0.5
	CC58	126	39	13.9	85.0	30.0	90	5.8	5.8	0.0	-	-	-	0.7	0.2
	H106	140	29	12.5	72.0	90.0	208			0.0	-	-	-	0.8	0.7
	CC59	159	40	11.0	110.0	208.0	60	2.2	2.2	0.0	-	-	-	0.9	1.5
	H107	88.7	52	19.7	80.0	60.0	175	-	-		-	-	-	0.5	0.4
	H108	235	20	7.4	80.0	175.0	136	-	-		-	-	-	1.3	1.3
	H109	78	34	22.4	46.0	136.0	100	-	-		-	2	1	0.4	1.0
						100.0									0.7
	H110	193	23	9.0	78.0	50.0	100	-	-		-	-	-	1.1	0.5
	H111	219	18	8.0	67.0	100.0	211	-	-		-	2	-	1.3	1.0
38	CC60	92.1	38	19.0	61.0	211.0	33	2.2	2.2	0.0	-	1	1	0.5	2.1
						33.0									0.3
	H112	65.1	53	26.8	60.0	146.0	155	-	-		-	-	-	1.0	1.1
	H113	71.6	34	24.4	43.0	155.0	163	-	-		-	-	-	1.1	1.1
39	H114	64	43	27.3	48.0	163.0	85	-	-		-	1	-	1.0	1.2
						85.0									0.6
40	CC61	76.3	30	22.9	40.0	140.0	70	3.7	3.7	0.0	1	2	1	0.5	1.2
	CC62	110	34	15.9	65.0	70.0	105	2.1	2.1	0.0	-	2	-	0.7	0.6
	CC63	87.8	34	19.9	52.0	105.0	40	4.8	4.0	0.8	-	1	1	0.6	0.9
	CC64	54.6	52	32.0	50.0	40.0	125	-6.1	-6.1	0.0	-	-	-	0.3	0.3
	H115	209.8	22	8.3	80.0	125.0	55	-	-		-	-	-	1.3	1.0

	H116	93.3	40	18.7	65.0	55.0	270	-	-		-	2	-	0.6	0.5
	H117	475	21	3.7	175.0	270.0	168	-	-		-	2	1	3.0	2.2
						168.0									1.4
	H118	32.4	74	53.9	42.0	20.0	103	-	-		-	2	1	0.5	0.4
	H119	99	32	17.6	55.0	103.0	70	-	-		-	-	-	1.5	2.0
	CC65	39.7	115	44.0	80.0	70.0	40	3.5	3.5	0.0	-	1	-	0.6	1.4
41	CC66	101.6	39	17.2	70.0	40.0	20	4.3	4.3	0.0	-	-	1	1.5	0.8
						20.0									0.4
	H120	148.8	40	11.7	104.0	64.0	162	-	-		-	1	-	0.6	0.5
	H121	400	24	4.4	170.0	162.0	100	-	-		-	1	-	1.5	1.3
	V36	-			185.0	100.0	225	3.9	-1.2	5.1	-	1	-		0.8
42	H122	247	27	7.1	115.0	225.0	75	-	-		-	1	1	0.9	1.8
						75.0									0.6
	V37	-			285.0	120.0	175	6.4	-1.4	7.8	-	2	1		0.9
43	H123	283	34	6.2	170.0	175.0	100	-	-		1	-	-	1.0	1.3
						100.0									0.8
	H124	59.2	59	29.5	61.0	30.0	173	-	-		-	-	-	0.7	0.3
44	H125	101.5	38	17.2	67.0	173.0	69	-	-		-	1	-	1.3	1.9
						69.0									0.8
45	H126	65.3	48	26.7	55.0	32.0	133	-	-		-	-	-	0.9	0.4
	H127	85.7	42	20.4	63.0	133.0	160	-	-		1	1	-	1.1	1.5
	H128	77	46	22.7	62.0	160.0	25	-	-		-	-	-	1.0	1.8
						25.0									0.3
46	H129	50.7	62	34.4	55.0	20.0	175	-	-		-	1	-	0.5	0.2

	H130	115.7	33	15.1	67.0	175.0	80	-	-		-	-	-	1.2	2.2
	H131	57.3	50	30.5	50.0	80.0	115	-	-		-	-	-	0.6	1.0
	CC67	53.4	59	32.7	55.0	115.0	15	-2.8	-2.8	0.0	-	-	1	0.6	1.4
	CC68	193.6	25	9.0	86.0	15.0	82	-2.8	-2.8	0.0	-	1	-	2.1	0.2
						82.0									1.0
	H132	29.3	100	59.6	51.0	20.0	200	-	-		-	1	1	0.4	0.3
	H133	120	48	14.6	100.0	200.0	116	-	-		-	-	-	1.7	2.6
	CC69	42.7	67	40.9	50.0	116.0	50	2.8	2.8	0.0	-	-	1	0.6	1.5
	CC70	35.3	84	49.5	52.0	50.0	76	-2.7	-2.7	0.0	-	1	-	0.5	0.7
	AH35	26.5	65	65.9	30.0	76.0	0	-	-		-	-	-	0.4	1.0
	H134	120	21	14.6	45.0	0.0	115	-	-		-	1	-	1.7	0.0
47	H135	128.4	29	13.6	65.0	115.0	30	-	-		-	1	-	1.8	1.5
						30.0									0.4
	H136	112	24	15.6	46.0	100.0	11	-	-		-	-	-	0.6	0.9
	AH36	146	18	12.0	46.0	11.0	45	-	-		1	-	-	0.8	0.1
	V38	-			210.0	45.0	125	4.7	-2.8	7.5	-	-	-		0.4
	H137	221	31	7.9	120.0	125.0	150	-	-		-	-	1	1.2	1.1
	H138	278	27	6.3	130.0	150.0	46	-	-		-	-	-	1.5	1.3
	H139	274	26	6.4	123.0	46.0	300	-	-		-	-	-	1.5	0.4
	H140	153	72	11.4	193.0	300.0	0	-	-		-	-	-	0.8	2.7
48	H141	128	58	13.6	130.0	0.0	225	-	-		-	-	-	0.7	0.0
						225.0									2.0
49	H142	73.8	47	23.7	60.0	60.0	155	-	-		-	-	-	0.5	0.5
	H143	77.2	49	22.6	66.0	155.0	145	-	-		-	2	-	0.5	1.3

	H144	215	24	8.1	89.0	145.0	150	-	-		-	1	-	1.5	1.2
	H145	200	25	8.7	87.0	150.0	88	-	-		-	1	-	1.4	1.3
						88.0									0.7
	H146	49.2	48	35.5	41.0	208.0	70	-	-		-	-	-	0.3	2.0
	H147	108.3	42	16.1	80.0	70.0	102	-	-		-	-	-	0.6	0.7
	H148	76.8	27	22.7	36.0	102.0	82	-	-		-	-	-	0.5	1.0
	AH37	48.3	55	36.2	46.0	82.0	75	-	-		-	2	-	0.3	0.8
	H149	121.1	26	14.4	55.0	75.0	115	-	-		-	-	-	0.7	0.7
	V39	-			235.0	115.0	50	3.3	-4.7	8.0	-	1	-		1.1
50	H150	597	11	2.9	115.0	50.0	110	-	-		1	2	-	3.6	0.5
						110.0									1.1
	H151	273.5	17	6.4	80.0	82.0	125	-	-		1	1	-	1.0	0.6
	H152	366.1	14	4.8	87.0	125.0	138	-	-		-	-	-	1.3	0.9
	AH138	176.1	39	9.9	120.0	138.0	140	-	-		2	-	-	0.6	1.0
	H153	612	12	2.9	131.0	140.0	284	-	-		-	1	-	2.3	1.0
	H154	130.5	33	13.4	75.0	284.0	100	-	-		-	1	-	0.5	2.1
51	H155	73.7	33	23.7	43.0	100.0	95	-	-		-	-	1	0.3	0.7
						95.0									0.7
	H156	53.6	59	32.6	55.0	112.0	25	-	-		2	1	-	0.6	1.9
	CC71	51.6	78	33.8	70.0	25.0	30	-	-		-	2	-	0.6	0.4
	CC72	140.3	26	12.4	64.0	30.0	32	-3.0	-3.0	0.0	-	-	-	1.6	0.5
	H157	118.3	28	14.8	57.0	32.0	100	-	-		-	1	-	1.3	0.6
52	CC73	78.9	54	22.1	75.0	100.0	50	-	-		-	1	-	0.9	1.7
						50.0									0.9

53	H158	122.5	47	14.3	100.0	122.0	160	-	-		1	-	-	0.8	1.2
	H159	87.7	59	19.9	90.0	160.0	110	-	-		-	-	-	0.5	1.6
	AH39	137	26	12.7	61.0	110.0	50	-	-		-	-	-	0.9	1.1
	H160	261.5	26	6.7	120.0	50.0	170	-	-		-	-	-	1.6	0.5
	H161	189	44	9.2	146.0	170.0	18	-	-		-	-	-	1.2	1.7
	CC74	163	27	10.7	77.0	18.0	76	-2.7	-2.7	0.0	1	-	-	1.0	0.2
						76.0									0.8
	H162	141.7	26	12.3	65.0	100.0	45	-	-		-	-	-	1.1	1.2
	AH40	119.4	29	14.6	60.0	45.0	20	-	-		-	-	-	0.9	0.5
	H163	72.2	44	24.2	55.0	20.0	130	-	-		-	-	-	0.5	0.2
	H164	95.4	57	18.3	95.0	130.0	25	-	-		-	-	-	0.7	1.5
	CC75	31.9	81	54.7	45.0	25.0	24	-2.3	-2.3	0.0	-	1	-	0.2	0.3
	H165	167.3	27	10.4	79.0	24.0	135	-	-		-	-	-	1.2	0.3
	H166	250.9	23	7.0	100.0	135.0	230	-	-		-	-	-	1.9	1.6
54	H167	194	25	9.0	85.0	230.0	57	-	-		-	1	-	1.4	2.7
						57.0									0.7
	H168	30.5	71	57.3	38.0	30.0	126	-	-		-	1	-	0.2	0.3
	H169	222.4	12	7.9	46.0	126.0	10	-	-		-	-	-	1.4	1.4
	V40	-			140.0	10.0	100	2.8	-2.1	4.9	-	-	-		0.1
	H170	154.9	31	11.3	84.0	100.0	115	-	-		-	-	-	1.0	1.1
55	H171	210.4	28	8.3	101.0	115.0	160	-	-		1	1	-	1.4	1.3
						160.0									1.8
	H172	109.9	31	15.9	60.0	20.0	210	-	-		1	1	-	0.5	0.2
56	H173	373.8	20	4.7	129.0	210.0	140	-	-		-	1	-	1.8	1.6

	H174	264.6	16	6.6	75.0	140.0	245	-	-		-	1	-	1.2	1.1
	H175	176.9	31	9.9	95.0	245.0	16	-	-		-	-	-	0.8	1.9
	H176	138.9	28	12.6	68.0	16.0	142	-	-		-	-	-	0.7	0.1
						142.0									1.1
	H177	79.5	25	22.0	35.0	59.0	102	-	-		-	-	-	0.3	0.6
	H178	158.4	20	11.0	54.0	102.0	60	-	-		1	1	-	0.6	1.0
57	CC76	557.6	10	3.1	95.0	60.0	195	3.4	3.4	0.0	-	1	-	2.1	0.6
						195									1.9

LIST OF PUBLICATIONS

Refereed Journals (National/International)

National Journal

1. Ravi Shankar A. U., Anjaneyulu M.V.L.R., and **Sowmya, N.J(2013)** "Consistency evaluation of horizontal curves on rural highways**"**, Journal of Indian Roads Congress, IRC Journal Vol.73-4, pp91-99.

This paper is selected for presentation at IRC 74th annual convention at Guhavati-Assam on Janaury 2014.

2.**Sowmya N.J,** Ravi Shankar A. U., and Anjaneyulu M.V.L.R **"**"Consistency evaluation of vertical curves on rural highways**",** Journal of Indian Roads Congress, IRC Journal (Accepted for publication).

International Journal

3. **Sowmya, N.J,** Ravi Shankar, A. U. and Anjaneyulu, M.V.L.R. (2012). "Modelling Operating Speed and Speed Differential on Intermediate Lane Rural Roads". *International Journal for Earth Sciences and Engineering*, Volume-05, No. 05(01), ISSN 0974-5904, October, 2012, pp.1408-1414.

International Conferences

1. Sowmya, N.J, Ravi Shankar, A. U**.,** Lekha, B.M. "Accident data analysis of Intermediate Lane Roads in Dakshina Kannada district". (Accepted and to be presented), *International Engineering Symposium (IES-2013)* at Kumamoto University, Japan, March 4-6,2013.

National conferences

2. Ravi Shankar, A. U. and **Sowmya, N.J**. (2011). "Speed prediction models for Two-Lane Rural Highways". Proc. of *National conference on ' Recent advances in Civil Engineering Research*', February,2011, Karpagam University, Coimbatore.

3. Ravi Shankar, A. U., Anjaneyulu, M.V.L.R. and **Sowmya, N.J. (2011). "**Speed Prediction models for Horizontal curves on Rural Highways- a new measure". Proc. of *National conference on'Innovations in Civil Engineering (NCICE'11)*', March, 2011, MPNMJ Engineering College, Chennimalai.

BIO DATA

1	**Name**	**SOWMYA N.J**
2	**Date of Birth**	22-02-1977
3	**Qualification**	B.E in Civil Engineering M. Tech (Geo Technical Engineering)
4	**Experience**	12 years of Teaching experience in K.V.G.College of Engineering , Sullia. D.K. Karnataka.
5	**Permanent Address**	SOWMYA N.J. W/o. Shyama Prasad A.D. Addanthadka House Jalsoor Post, Sullia Dashina Kannada, Karnataka, India 08257-288883, 288205. Mob: 9448725762 Email: sowm.shyam@gmail.com

6 Academic Details:

Examination Passed	Year of Passing	University	Class
BE(Civil Engineering)	1998	Mangalore University	First Class with Distinction
M Tech (Geo Technical Engineering)	2000	N.I.T.K,Surathkal	First Class with Distinction
D. Info. Tech	2003	SMU-open university	First Class with Distinction

7. Subjects studied during M.Tech(GT)

1. Basic Geo-Mechanics	6. Advanced Design of Pavement
2. Soil Dynamics	7. Site Irrigation and Ground Improvement Technique
3. Geo-technology	8. Rock Mechanics
4. Finite Element Analysis	9. Earth and Rock fill Dams
5. Numerical Analysis	10.Analysis and Design of Foundations

8. Subjects studied during PhD course work

1. Traffic Engineering and Management
2. Road Safety and Traffic Management
3. Environmental Impact Assessment
4. Highway and Airport Geometric Design

9. Subjects Taught in UG Level

1.Geo Technical Engineering	6. Building Construction
2.Foundation Engineering	7. Highway Geometric Design
3.Ground Improvement Technique	8.Transportation Engineering
4. Fluid Mechanics	9.Irrigation Engineering
5.Hydraulic and Hydraulics Machinery	10. Pavement Materials and Construction

10. Professional Body Membership

1. Life member of Indian Society for Technical Education(ISTE)
2. Member of Indian Roads Congress (IRC)

11. Publications

National/ International journals -02

National/ International Conferences -03