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# Development of GPS-Based Procedure for Tracking Vehicle Path

Submitted by
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A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment of the requirements for the degree of Master of Applies Science

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*Your file Votre référence*
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*To my parents and my wife.*

# ABSTRACT

This thesis presents a GPS-GIS-based procedure to obtain the horizontal and vertical alignments of a road based on the path of a control vehicle. Using differential GPS surveying, field data were collected at a 0.1-second observation interval, under different speed conditions on two different sections of two-lane rural highways in eastern Ontario. The raw GPS data were post-processed to filter out possible errors and then imported into a GIS environment for analysis and interpretation. An extension for ArcView was written to determine the geometric features of the highway horizontal alignment, including the tangents, spirals, and circular curves. Values were obtained for the radius and length of seventeen circular curves, length of spirals, and the lateral position of vehicle path along the straight and curved segment. These were compared with the same features of the actual highway alignment. Results based on 0.1-second observation interval were better than those corresponding to higher observation intervals of 0.5-second and 1-second. The observed elevation data were converted to highway profiles using available software. During comparison with actual profiles, considerable difference was observed in absolute position of the fitted profiles whereas their relative position was very close to the actual ones. Highway segments with open surroundings gave better results than hilly or congested sections.

The results show that the developed procedure can produce both the horizontal and vertical alignment of a road quickly, accurately, and at a relatively low cost. In addition to the extraction of the alignment of a road, the procedure can be used to track the actual vehicle path under normal driving conditions and compare it with the horizontal alignment of a road in an investigation concerning driver behaviour.

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# List of Abbreviations and Acronyms

| | |
|---|---|
| AADT | Average Annual Daily Traffic |
| AASHTO | American Association of State Highway and Transportation Officials |
| abs. | Absolute |
| AM | After Meridian |
| A-S | Anti-spoofing |
| C/A code | Coarse Acquisition code |
| DOP | Dilution of Precision |
| DMTI | Desktop Mapping Technologies Inc. |
| km/h | Kilometres per hour |
| LoS | Loss of Signal |
| FHWA | Federal Highway Administration (USA) |
| FME | Feature Manipulation Engine |
| GIS | Geographic Information System |
| GPS | Global Positioning System |
| GUI | Graphical User Interface |
| ITS | Intelligent Transportation System |
| LT | Left Turn |
| MCS | Master Control Station |
| MB | Mega bytes |
| MHz | Mega Hertz |
| mm | Millimetre |
| MTO | Ontario Ministry of Transportation |

| | |
|---|---|
| NCHRP | National Cooperative Highway Research Program |
| NAD27 | North American Datum 1927 |
| *nps* | number of points to skip |
| ns | nanosecond |
| OD | Opposite Direction |
| OECD | Organization for Economic Cooperation and Development |
| PC | Personal Computer |
| P-code | Precise code |
| PI | Point of Intersection |
| ppm | Parts per Million |
| $R^2$ | Coefficient of determination |
| ROR | Run-off-Road |
| RRR | Resurfacing, Restoration, and Rehabilitation |
| RT | Right Turn |
| SD | Standard Deviation |
| SA | Selective Availability |
| SDG | Stormont Dundas and Glengarry |
| SV | Space vehicle |
| TCS | Transport System Centre |
| UTM | Universal Transverse Mercator |
| DoD | Department of Defence |
| Veh | Vehicles |
| WGS84 | World Geodetic System 1984 |

# Chapter 1

# INTRODUCTION

## 1.1 Background

The development of transportation systems has been a significant factor in the development of civilization as a whole. Our ability to move people and goods now seems virtually limitless when one considers, for example, the achievements of the various space programs. However, the success of a transportation system depends on its safety and reliability. Governments spend millions of dollars to maintain the transportation systems but one of the major concerns of the public is also their own safety. Over the past decade, safety has become a higher priority and a major standard by which the system is to be evaluated.

One of the unfortunate by-products of modern transportation is the price that society pays in terms of injury and loss of life. The safety problem in transportation encompasses all modes, including land, air, and water. However, about 95% of all transportation collisions are associated with highways (Sanderson, 1996). Highway safety is a worldwide problem; with over 500 millions cars and trucks in use, more than 500,000 people die each year in motor vehicle crashes, and about 15 million are injured (Garber and Hoel, 2001).

Motor vehicle crashes are also the leading cause of death in the 15 to 24 age group and the third leading cause of lost productivity (Sanderson, 1996). The estimated annual cost of highway collisions to the Canadian economy is at least $10 to 25 billion,

and 4% of the total Canadian health care costs in year 1993. In 2001, 2778 road users were killed in traffic collisions and almost 17,000 suffered serious injuries. Collectively, almost 224,000 road users, or more than 600 per day, became casualties who suffered some form of physical injury (Transport Canada, 2002).

There are many causes of highway collisions, which can be broadly grouped into three categories that are, driver, vehicle and road. Highway engineers have an effective control over only one of these elements—the roadway, which may contribute to collision occurrence through poor structural and/or geometric design. The geometric design of highways refers to the design of the visible dimensions of such features as horizontal and vertical alignments, cross section, intersections, and bicycle and pedestrian facilities. The main objective in geometric design is to produce a highway with safe, efficient, and economic traffic operations while maintaining aesthetic quality. Geometric design is influenced by the vehicle, driver, and roadway environment. The temporal changes of these characteristics make geometric design a dynamic field where design guidelines are periodically updated to provide more satisfactory design (Easa, 2003).

Highway alignment is one of the most important geometric features that can affect the highway's level of service and safety. Crash studies have indicated that roadway curves experience a higher crash rate, and a greater proportion of severe collisions than straight segments. It is estimated that more than 50% of the total fatalities on rural highways can be attributed to collisions that take place on curved sections (Gibreel et al., 2001). Glennon (1987) showed that the average collision rate for highway curves is about three times the average collision rate for highway straight segments and that the average single-vehicle run-off-road collision rate for highway curves is about four times that for

straight segments. A highway curve is one of the most complex but unavoidable features of all highways. The curve form has evolved from what appeared to be reasonable to the builder's eye to the more modern geometrically designed form of a circular curve with superelevation, cross slope transitions, and often spiral transitions.

It has been observed during various research studies that drivers do not track typical, simple circular curves as they are designed (Zegeer et al., 1991). Instead, a typical driver entering an unspiraled curve gradually follows a spiral or transition curve. At some point into the curve, the vehicle actually tracks a path sharper than the curve in order to avoid running off the road. Also, the frequency of centerline and edge-line encroachments increases as curvature increases, particularly for curves sharper than 5 degrees (350 meter radius) (Zegeer et al., 1991). This observation confirms the difficulty drivers have in tracking a circular curve at high speed.

In addition to horizontal curves, vertical curves with steep grades may also have higher collision rates than flat sections or sections with a mild grade. Similarly, downgrades are more hazardous than upgrades, especially for trucks. Driver sight distance is also an important factor in highway safety. On any point along the road, designers need to provide a sight distance sufficient for safe manoeuvring. The more sight distance provided, the safer the highway will be, if divers do not exceed the maximum speed limit. Also, another critical safety feature is roadside design, as this area is often encroached upon by the drivers during a collision. Clear shoulders, gentle side slopes, and the absence of solid objects give the driver time to recover.

Preventing collisions from occurring is the most effective means to improve highway safety. It is also the most difficult and complex task to accomplish. The causes

of traffic collisions are many and complex, but studies show that 96% of all collisions involve, among other factors, some form of driver error (McShane et al., 1990). Observing a driver's behaviour on road is therefore considered to be one of the important safety studies that guide road designers to improve the highway geometrics and make it according to the driver expectation within the available situation.

For improving safety of the highway system, transportation engineers must have accurate data on the location, frequency, severity, and types of collisions that occur. In order to develop corrective measures, detailed accident descriptions are required. The quality of such information plays an important role during studies on highway safety. The use of the Global Positioning System (GPS) offers the potential to collect appropriate data which can be used to improve transportation system design.

GPS is a full-time, all-weather, high-precision, satellite-based navigation system established in 1978 by the US Department of Defence, for military positioning applications and, as a by-product, has been made available to civilian applications (Czerniak et al., 1998). This system consists of a constellation of radio-navigation satellites, a ground control segment which manages satellite operation and users with specialized receivers that use the satellite data to satisfy a broad range of positioning requirements. Different types of GPS receivers are available on the market, and they can be used in various types of GPS surveying, depending upon the type/precision of data required.

## 1.2 Problem Definition

Highway curves continually show a tendency to be high-collision locations despite a reasonably well-conceived design procedure, which considers a tolerable level of lateral acceleration on the driver. The collision causal aspect of highway curves is one of the most complicated areas of highway safety. In this context, the most important factors are curve radius and length. But, both radius and length of curve are not totally independent of other elements that together contribute to collision occurrence. For example, the sharpest curves may be located on lower quality highways; those with narrow carriageways, narrow shoulders, marginal sight distance, hazardous roadsides, and the like.

Curves are designed based on the assumption that vehicles follow the path of the highway curve with geometric exactness. It has been found that at some point along the highway curves of large radii (250 to 875 m), the vehicle path curvature is more severe than that of the curve (Glennon et al. 1972). The discrepancy between minimum path radius and curve radius increases with increasing curve radius. Other researchers observed during their studies on vehicle lateral trajectories that, on low radius curves, drivers tend to let the vehicle shift in the lane so that the vehicle path curvature is less severe than that of the curve (McLean 1974).

To find the exact track followed by vehicles on curves, different techniques have been employed by researchers. For example, Glennon (1972) used photographic field studies of vehicle manoeuvres on highway curves. New developments in data collection procedures, notably computer associated data collection and automated data collection using GPS, should enable researchers to record travel behaviour in a more automated

way. The GPS automatically records the changes in position of a person or vehicle together with time information. GPS data collection could improve the quality of data, and provide data storage in digital formats, which would enable the direct analysis of the data.

## 1.3 Objectives

This study attempts to develop a standard procedure for tracking vehicle trajectory on highway curves with a high level of positional accuracy using GPS. The focus of the thesis includes:

- Application of GPS technology for tracking vehicle trajectory on two-lane rural highways to develop a time-saving, simple procedure to collect highway data. The procedure can also be used to determine the existing highway horizontal and vertical alignment.
- Integration of the GPS and Geographic Information System (GIS) for analysis of the positional data.
- Development of a computer program in a GIS environment to find the various geometric features of highway curves using the geometrical relationships between the GPS coordinates, radius of curvature and the azimuth.
- Establishment of highway horizontal and vertical alignment using three-dimensional coordinates' field data.
- Determination of the lateral displacement of vehicle from the actual alignment.

## 1.4 Thesis Organization

This thesis is comprised of seven chapters. Chapter1 introduces the background to the subject and elaborates on the objectives of the study after identification of the problem. Chapter 2 presents a review of the research work conducted in relation to highway alignment and safety. Chapter 3 provides an overview on GPS and its components, techniques and sources of error. A brief discussion on GIS is also presented, followed by a review of the available literature on the application of these technologies in transportation engineering research. Chapter 4 describes the procedure adopted to collect the field data and the post-processing methods. Chapter 5 presents the analysis of the horizontal alignment focusing on the development of the proposed software and sensitivity analysis for establishing its parameters. Chapter 6 presents the data analysis for the vertical alignment of the highway using the already established software. Chapter 7 contains a summary of the work done, conclusions, and recommendations for future work. An outline of the references used in the thesis is provided at the end followed by appendices for supporting the analytical work.

# Chapter 2

# ALIGNMENT GEOMETRY AND SAFETY

## 2.1 Introduction

Transportation has historically focused on capacity and congestion with some attention to the operation and management of the transportation system. Over the past decade, safety has become a higher priority and a major standard by which the system is to be evaluated. The goal of highway safety research is to promote public health and safety by working toward the elimination of highway related deaths, injuries and property damage. Although implementing the safety goals is a challenging job, progress to improve road safety has been made. In Canada, since 1984, fatalities resulting from traffic collisions have decreased by 33% and serious injuries have declined by 35% (Transport Canada, 2002). These positive changes have come about despite substantial increases in the population (26%), in the number of licensed drivers (34%) and in the number of motor vehicles registered (26%) (Transport Canada, 2002). However, in spite of these dramatic improvements, deaths and injuries resulting from traffic collisions continue to be the major transportation safety problem in Canada. In 2001, 2,778 road users were killed in traffic collisions and almost 17,000 suffered serious injuries (defined as requiring hospitalization for 24 hours or more).

Transportation professionals are continuously working for providing maximum possible safety to the road user. Road safety research is the scientific study of the road and traffic system with the fundamental aim of finding ways of reducing the number of

road collisions or their severity (Foot et al., 1981). However, the task of reducing the number of collisions presents many technical problems, which require more than common sense and common knowledge for their solution. Therefore, research studies, using the techniques that have been developed in a scientific work, are considered essential if reliable information is to be obtained. In this field, where opinions are easily formed and strongly propagated, there is a great need for facts. Hence research studies about the road safety are usually based on the nationally collected collision records or on information collected for specific investigations.

Detailed research can be classified according to whether they concern principally the road, the road user, or the vehicle (Harmondsworth, 1963). The first two classes, the road and the road user, have been taken into consideration in setting out the following review of published literature, with emphasis on highway curves and the path followed by vehicles plying on the curves. An overview of the application of advanced technologies such as GPS and GIS is also presented. From a safety perspective, the roles of all three components (road, user, and vehicle) are interrelated.

## 2.2 Highway Collisions

Collision statistics for highways provide basic information on the relative importance of the various factors that contribute to collisions. They provide useful guides to possible measures of preventing road collisions and assist in determining the directions along which further investigations would be most useful. Collision statistics also show the outcomes of the measures adopted to combat the collision problem. They provide a record of the trends in the collision situation and are used to assess the collision

possibility of different classes of road users and different types of roads, road features and vehicles.

Every transportation system can be seen as comprised of three main elements: the human, the machine, and the environment. In terms of highways this translates into: the driver of the vehicle (in addition to passengers, pedestrians and other humans in and near the roadway), the motor vehicle as the basic machine, and the roadway and its traffic as the environment. A road collision is often the result of a person's and/or vehicle's inability to cope with the environment. There are three main approaches for reducing the risks of this occurring (Foot et al. 1981):

1. Change the environment so that the road user is physically guided into taking the correct action (Engineering).
2. Supply road users with information and practical training to cope with this environment (Education).
3. The application of traffic laws, which, while apparently restrictive in content, are, nevertheless designed to guide users safely around the road system (Enforcement).

The Organization for Economic Cooperation and Development (OECD) created an expert group to propose strategies that could reduce the number of deaths and injuries resulting from traffic collisions on rural roads (Hasson, 2000). It was found that road-user behaviour does not stand on its own; rather it is largely dependent on the road environment. Rural road collisions are generally more severe than collisions on urban

roads due to the differences in operating speeds, road geometry, functionality, enforcement levels and other factors. At the international level, as much as 80% of all collisions on rural roads fall into three categories: single vehicle collisions—especially running off the road (35%), head-on collisions (25%) and collisions at intersections (20%) (Hasson, 2000). Run-off the road collisions and head-on collisions are more common on curves than on the adjacent tangent sections. The most dangerous curves in either case are isolated curves and the first in a series of curves after a straight or almost-straight section. Two main causes of collisions on curves were determined as (Hasson, 2000):

- Loss of control due to inappropriate speed while negotiating the curve,
- Improper assessment of the situation due to limited sight distance on curve.

Traffic collisions are the measure for gauging road safety in both rural and residential areas (Foot et al., 1981). However, attempts to estimate the relative safety of locations are usually problematic because of the unreliability of records or the randomness of collisions and the time required to accumulate adequate sample sizes. Hence the traffic conflict technique was developed to measure traffic hazards at particular sites. In practice, however, the use of traffic conflicts as a measure of safety gives rise to a number of problems. These include difficulties of defining 'conflicts' and 'severity' and problems of reliability and validity (Foot et al. 1981).

## 2.3 Highway Alignments and Safety

Safety has been of interest to transportation engineers for many years. It was realized long ago that among considerations of mobility, convenience and economy of operation, safety is of primary concern. Through various research studies, a strong association between adverse highway geometry and collision occurrence was also established (Garcia et al., 2000). It can be said that improvement in specific highway geometric features can significantly reduce the occurrence or severity of highway crashes, and thus significantly contribute to the economic justification of highway investments. The safety benefits of many design features however are not well known primarily due to the lack of detailed data to link the alignment characteristics at specific sites to collision data.

Keeping highway safety in mind, a highway designer working on the plans for a new highway will always strive for gentle highway alignment consisting of flat horizontal curves, non-critical grades, and long vertical curves (Glennon, 1987). In the United States and some other countries, during the last 60 years, this process has been guided by the design policies of the American Association of State Highway and Transportation Officials (AASHTO), which has defined acceptable limits on these design features based on perceived safety and operational effects (Glennon, 1987). Although, cost effectiveness has always been an underlying basis for design, these design limits have been largely governed by acceptable performance criteria rather than cost effectiveness considerations.

The driver is the most variable element in the road system and has the greatest effect on the random component of road collisions. Yet, it was observed that collisions involving isolated vehicles are often associated with a possible defect in the alignment

(Cafiso, 2000). In a rural environment, and in free flow conditions, the elements that influence vehicle speed are essentially the standard of the highway link (one or more lanes, single or dual carriageway), the gradient of the road and, above all, the degree of curvature and the horizontal alignment. The study also revealed that although the operating speed in the central section of the curve is comparatively the lowest, the effective trajectory of the vehicle at this point reaches the tightest curvature, and this is where the most critical conditions for equilibrium occur.

Although, the approaching speed is the main factor that affects the driver's choice of path and speed to travel curves with the same radius, there could be other relevant factors to be considered, particularly those affecting the perception of curvature. One of them could be the spiral curve used in the transition sections. Spiral curves help in providing a gradual increase in centrifugal force, superelevation, and a satisfactory highway appearance (Crisman et al., 2004). The influence of the spiral on safety has been analyzed in a limited number of studies reaching different conclusions. Some of these results seem to show a possible negative effect of spiral transition on the driver curve perception and safety (Crisman et al., 2004). Consequently, some road standards have taken into account these results by limiting the spiral length, at least on sharp curves requiring considerable speed reduction (Crisman et al., 2004).

The relationship between highway alignment and safety was studied by O'Cinneide (1998) and it was reported that a road alignment inconsistency such as an isolated narrow curve in an otherwise straight alignment is more dangerous than a succession of curves of the same radius. Also, horizontal curves are more dangerous when combined with gradients and surfaces with low coefficients of friction. The

increase in collision rates becomes particularly significant at radii below 200 m. More recent work suggests that the impact of transition curves is neutral (O'Cinneide, 1998). The geometry of vertical curves is not known to have a significant effect on collision severity. It has also been investigated that the horizontal realignment of rural highways with improved design features is the most efficient way of increasing safety; reductions in the number of collisions of the order of 80 percent have been reported in this context (O'Cinneide, 1998).

In another study on highway alignment, conducted by Hasson (2000), it was recommended that a series of relatively wide curves should not be followed by a very narrow one without extensive warning and/or physical speed reduction measures. The distance between two successive curves or between a straight section and a curve should also be long enough—three second driving time—to allow drivers to judge and interpret the situation. It was admitted that straightening horizontal curves, though effective, is expensive and probably only cost-effective on higher-volume roads (Hasson, 2000). Less expensive measures which may help reduce collision rates on curves include the removal or protection of roadside hazards, flattening side slopes, improving skid resistance, increasing superelevation, paving the shoulders and eliminating pavement edge drops. Typical low-cost measures include upgrading the pavement edge lines and centerlines, adding raised reflective pavement markers or upgrading the advance warning.

Many existing highways in North America were originally small roads that were built without following specific engineering standards. Consequently, the vertical alignment of these highways usually follows a curvilinear profile with no clear tangents or curves (Easa et al., 1998). The resulting vertical curves do not usually follow a specific

mathematical formula, and are referred to as spline grades. The profiles of such vertical alignments contain a number of station points and their corresponding elevations recorded during field surveys. Easa et al. (1998) developed an analytical method for establishing the vertical alignment of a highway section using profile field data. They developed computer software using the least squares method of curve fitting to determine the straight and curved segments and estimate the elevation, the instantaneous grade and the instantaneous rate of vertical curvature. The method includes identification of critical points (points that lie between two successive curves or a straight segment and a curve) and the segment shape (straight or curved) between each two critical points and then fitting straight lines and spline curves to the data points within the straight and curved segments, respectively. The advantage of this method is the reduction in time and effort required for vertical alignment preparation as compared to the current practice of graphical methods.

## 2.4 Highway Cross-Section and Safety

While studying the effect of lane width, shoulder width, and shoulder type on highway safety, Zegeer and Deacon (1987) reviewed more than thirty articles and reports dated between the early 1940s and the mid-1980s. They reported that the conclusions of these studies were often not only inconsistent, but also, in many cases, totally contradictory. For example, some studies concluded that wider shoulders result in an increased number of collisions, whereas others found that shoulder width had little or no effect on collisions. Because of this disparity in research findings, considerable selectivity was demanded in determining which studies should be considered among the

most reliable. Of the more than thirty articles reviewed, nine studies were deemed most appropriate for detailed consideration, and information from four of the nine was ultimately used in developing the most likely collision relationships (Zegeer et al., 1987).

The collision types found to be most related to lane and shoulder widths and shoulder type were Run-off-Road (ROR) and Opposite-Direction (OD) collisions. Though, these studies could not develop a satisfactory quantitative model for estimating the effect of lane width, shoulder width, and shoulder type on collisions on two-lane rural highways. General effects of these elements were qualitatively summarized as (Zegeer et al., 1987):

- Lane and shoulder conditions directly affect ROR and OD collisions. Other collision types, such as rear-end and angle collisions, are not directly affected by these elements.
- Rates of ROR and OD collisions decrease with increasing lane width up to 11 feet (3.35 m). However, the rates remain approximately the same (or slightly higher) for 12-feet (3.65 m) lanes, possibly indicating the limit beyond which further increase in lane width is ineffectual.
- Rates of ROR and OD collisions decrease with increasing shoulder width up to 10 feet (3.05 m). However, the effect of shoulder width increments on rate of collisions is diminished beyond this limit.
- Lane width has a greater effect on collision rate than shoulder width
- Non-stabilized shoulders, including those constructed of loose gravel, crushed stone, raw earth, and turf, exhibit larger collision rates than

stabilized (e.g., tar with gravel) or paved (e.g., bituminous or concrete) shoulders.

## 2.5 Driver's Path and Safety

Among many other parameters, vehicle lateral placement on curves is considered to be the most significant evidence of unsafe traffic conditions (Steyer et al., 2000). Using car following techniques, it was observed that the driving path often does not coincide with the circular arc forming a horizontal curve. Consequently all discussions about dynamic driving conditions on a curve should consider the driving path. Analysis of the research results revealed that the radius of the curve and the deflection angle of the circular curve would be appropriate parameters for the assessment of the driving path. The ratio of the deflection angle to the radius of the horizontal curve was found to have the highest coefficient of determination ($R^2$ = 0.83 for both left and right curves) and therefore the strongest relationship to the driving path of the curve. Lateral placement of vehicles traversing a curve was categorized as good, critical, dangerous and very dangerous depending upon whether driving in their own lane, encroaching centerline or edge lines, overlapping of opposing driving area with a sufficient opposing sight distance and overlapping of opposing driving areas without a sufficient opposing sight distance, respectively. Low cost countermeasures were found less effective for the last two categories; whereas RRR-projects (Resurfacing, Restoration, and Rehabilitation) were considered more successful.

Garcia and Diaz (2000) used image processing techniques to measure the variables that define driver behaviour in terms of lateral placement, path curvature, speed

and side friction demand. They carried out a field trial on a curve in order to compare the observed vehicle behaviour with the assumed one in the road geometric design guides. A considerable variation was found between individual drivers in the way they negotiated the curve. Also, the profiles showed a strong tendency for vehicles to cut the corner, as evidenced by the change in lateral placement around the curve, especially at the centre of the stretch. A great discrepancy was revealed between the radii of the paths followed by drivers and the curve radius. There were some positions where path curvature was higher than the centerline curvature.

To investigate possible safety conflicts between the driver and the road, a new approach was developed at the Dresden University of Technology (Steyer et al., 2000). Using the "driving conflict technique", they included three different levels to be weighted for evaluating the safety level of two-lane rural roads. These levels are:

1. Quality of the horizontal and vertical alignments as well as the cross section and the road appearance expressed by the three dimensional alignment.
2. Level of service.
3. Driving dynamic relationships between the vehicle and the road.

Based on the results of comprehensive practical measurements of the driving behaviour, they also developed a procedure to analyze and improve the safety level of curves in the two-lane rural highways. The procedure included three components; collision analysis model, curve geometric model and curve driving behaviour model. Based on the analysis of the results of these three models, an additional component,

called "measurement set model" was also developed to select appropriate low cost countermeasures to increase the traffic safety on two-lane rural roads.

Besides identifying significant relationships between road collisions and road geometric design, a substantial proportion of the research studies suffered from methodological problems (Garcia et al, 2000). Thus one can say that the relationship between collisions and design features is still not known precisely. Other researchers have also criticized the geometric road design standards for providing unknown levels of safety. In this context, Felipe and Navin (1998) obtained statistical information on the basic variables involved in driving through a horizontal curve. Data were collected on a two-lane rural highway passing through mountains, with both normal and expert drivers as well as a passenger sitting with the driver to observe the comfort level. The results indicated that, for a comfortable ride, drivers are limited by their lateral acceleration on small radius curves and seek the "environmental speed" on large radius curves. On large radii curves, car-driver systems mostly followed the center of the lane for each direction. For the smaller radii, drivers in both directions "cut" the curves. To minimize the speed change, drivers flattened the bends either by driving on the shoulder or, in some cases, by driving partly in the opposite lane.

Glennon et al. (1972) conducted photographic field studies, simulation studies, and controlled experiments on vehicle manoeuvres on highway curves. Their aim was to examine the adequacy of the current geometric design's assumption that vehicles follow the path of the highway curve with geometric exactness. They recorded vehicle paths on film using a camera housed in a following vehicle. The results of their study revealed that most of the vehicle paths, regardless of speed, exceed the degree of highway curve at

some points on the curve. Based on the results of that study, they proposed a new design approach. This approach is dependent on selecting:

1. An appropriate vehicle path percentile relation.
2. A reasonable safety margin to account for unexplained variables that may either raise the lateral friction demand or lower the available skid resistance.
3. A minimum skid resistance versus speed relation that the highway department will provide on all pavements.

Blana and Golias (2002) carried out a study to investigate the vehicular lateral displacement while driving on curved and straight rural road sections in real-road and simulator conditions. They used ground-level and elevated cameras and observed free flowing vehicles only. The statistical analysis of the data revealed that differences in values of the lateral displacement between the two environments were generally higher at points where lateral position could be confined because of poor road geometry and visibility, such as the entry, apex, and exit of the curves. On both straight and curved portions of the road, the real road drivers drove considerably closer to the center of the road compared with simulator drivers. The average standard deviation of the lateral displacement in the simulator was at least double that on the real road for five of the seven points considered for all curve driving speeds. On straight sections, real-road drivers kept a more or less constant distance from the edge of the road (about 1 m), whereas simulator drivers kept moving closer to the edge of the road.

While studying the effect of speed on lateral displacement, it was observed that at lower speeds on curves (less than 60 km/h), lateral displacement differences between the two environments were considerable because the simulator participants positioned their vehicle closer to the edge line compared to the real-road drivers. The differences in lateral displacements on curves between the two environments decreased at higher speeds (more than 60 km/h), as the simulator participants' feeling of safety decreased more quickly than that of the real-road drivers. On straight sections, as the speed increased, the real-road drivers increased their distance from the edge of the road due to their fear of unexpected hindrances; but the simulator drivers did not appear to perceive this feeling of risk.

Usually the driving behaviour on curves of two-lane rural highways is described by speed. A study was carried out by Spacek et al. (2000) to consider the possibility that track behaviour (path of vehicle) is more suitable to distinguish unconscious or unintentional failures made by drivers negotiating curves. After defining six track types with their respective features, the frequency of occurrence of individual track types and their correlations with curve geometry were evaluated. Then the correlations between track behaviour and collisions in curves were examined. It was found that different patterns of track paths do exist on curves, and that the frequency of the individual track types differs considerably from curve to curve. The analysis also showed that relations exist between collision frequency and frequency of certain track types. One important finding of this study is the fact that the cause of the single car collisions, frequent on curves, cannot be solely attributed to inappropriately high speeds. According to the measurements made in the study, it must be assumed that in certain track types primarily

the steering corrections made by the drivers are responsible for locally increased centrifugal accelerations.

One of the reasons for adverse steering actions was found to be the increased length of spiral curve (Crisman et al., 2004). It was observed that approximately 1-2 seconds before entering the curve, drivers move the fixation point of their look to the inside edge of the curve and use the visual information collected a few seconds ahead in the anticipatory mode. If the spiral is too long, the driver looks at the first part of the transition that has an average curvature lower than the following circular arc and uses this information to decide the steering action and speed. Moreover, the steering action ends on the spiral because the steering time is smaller than the real transition length. Therefore, the driver, after the end of the planned steering action, has to make some unexpected steering and speed correction in order to follow the inner line and its increasing curvature along the spiral. The need of these unexpected corrections after the end of the planned steering manoeuvre results in an increase in the driver workload and, especially with high approaching speed and low side friction available, could cause the loss of control of the vehicle. Based on the kinematic analysis of a vehicle's lateral motion along the transition action, it was observed that the most desirable operating conditions are obtained when the spiral length is approximately equal to the length travelled at the operating speed during the steering time (Crisman et al., 2004).

Based on the results of the same study (Crisman et al., 2004) it was concluded that longer spirals should be used only before large radii when there is no need to reduce the speed, and consequently the perception of the correct curvature would not be so important.

## 2.6 Summary

Review of the available literature on the relationship between highway alignment and traffic safety clearly indicates that there is still enough room to improve the current geometric design guidelines. In this context, one of the most important areas is to precisely determine the actual path followed by a vehicle while travelling at the design speed on a highway. The difference between the vehicle path and the actual alignment of the highway may be a good indicator of the design discrepancy and at the same time it may guide the researchers to further improve the design procedure. Moreover, there is need for a simple and efficient procedure to track the existing alignment of highways to carry out any improvement required according to revised design values. The current study aims to develop a single procedure which can be used both for tracking vehicle path as well as highway alignment.

# Chapter 3

# THE GLOBAL POSITIONING SYSTEM

## 3.1 Overview

The GPS is a satellite-based radio-navigation system that was developed by the U.S. Department of Defence (DoD) in the early 1970s. Initially, GPS was developed as a military system to fulfill U.S. military needs. However, later on it was made available to civilians, and is now a dual-use system that can be accessed by both military and civilian users. GPS provides continuous (24-hour) positioning and timing information, anywhere in the world under any weather conditions. For security reasons, GPS is a one-way-ranging system, in which users can only receive the satellite signals. Navigation, surveying and integration with GIS are just a few examples of the successful application of GPS technology.

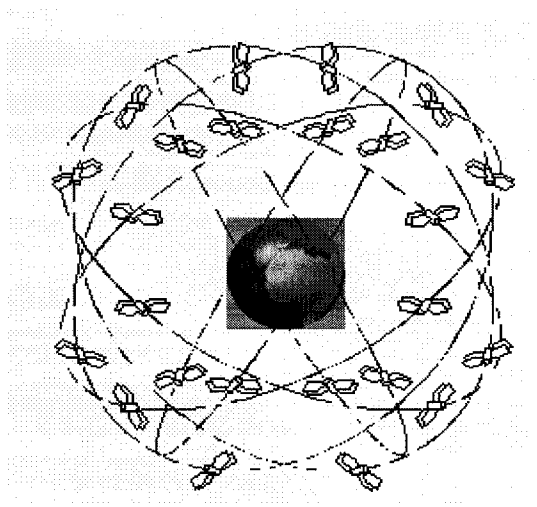


**Figure 3.1: Constellation of Satellites.**

[Source: Leica Geosystems 2002]

GPS receivers acquire signals from a constellation of satellites to determine position. By logging position and time data, other kinematic parameters, such as velocity and acceleration can be derived. The basic principle of GPS is:

$$\text{Range} = \text{Time taken} \times \text{Speed of light}$$

GPS consists, nominally, of a constellation of 24 operational satellites (Figure 3.1). To ensure continuous worldwide coverage, GPS satellites are arranged so that four satellites are placed in each of six orbital planes. With this constellation geometry, four to ten GPS satellites will be visible anywhere in the world, if an elevation angle of $10^{\circ}$ is maintained (El-Rabbany, 2002). Elevation angle is the angle from the GPS receiver's antenna between the horizontal line and the line of sight to the satellite (GPS positioning guide, 1995). Only four satellites are needed to provide the positioning information. GPS can provide a wide range of accuracies, depending on the type of measurements used and procedures followed. In general, the higher the accuracy required, the higher the cost and the greater the complexity of using the GPS.

In terms of spatial information system, positional accuracy refers to how closely the data represent a feature of the real world. In other words, accuracy is the closeness of an estimated value of a dimension to its actual or true value. Accuracy is always used as a relative measure, because it is always measured relative to some specification (Fekpe et al., 2003). In analogy with accuracy, another term – precision – is also commonly used in carrying out measurements. Precision refers to closeness of an estimate to the mean value and the fineness of the measurement scale. In other words, precision characterises the degree of mutual agreement or repeatability among a series of individual measurements, values, or results. In this thesis, both terms – accuracy and precision – are used to indicate

the closeness of the fitted geometric parameters of the highway alignment to their actual values in the available plans and maps.

GPS is a complex system which can be used to achieve position accuracies ranging from 100 meters to a few millimetres depending on the equipment used and procedures followed (GPS positioning guide, 1995). Higher accuracies correspond with higher costs and more complex observation and processing procedures. It is important for users to understand what techniques are required to achieve desired accuracies with the minimal cost and complexity. To provide a standard upon which GPS surveys may be tested (and in particular high accuracy surveys), the Geodetic Survey Division, Canada in cooperation with provincial agencies, has established several GPS base nets across the country. Each of these base nets consists of six to eight stations marked with forced-centering pillars; with inter-station distances ranging from 2 to 50 km in most locations (GPS positioning guide, 1995). The Geodetic Survey Division may be contacted for information on Canadian base nets.

It should be noted that the GPS does not give altitudes as accurately as it gives the horizontal coordinates (Czerniak, 1998). Also, the standard deviations of heights can differ from standard deviations of horizontal components by a factor of 10 (Design survey manual, 2005). One of the reasons for this error is the geometry of the satellites. Theoretically, a fifth satellite located at the center of the earth would solve this problem. Lack of sufficient and reliable vertical control points and limitations in geoid models are also sources of error in vertical position.

## 3.2 GPS Components

The GPS is made of three segments: the space segment, the control segment and the user segment, as shown in Figure 3.2.

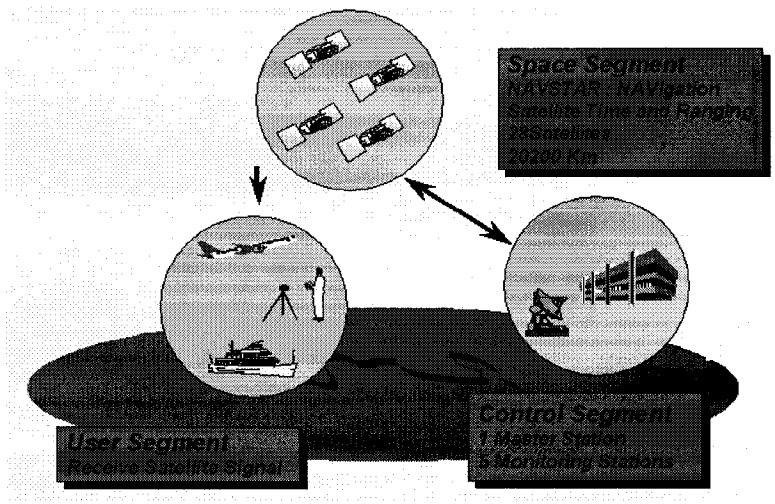



**Figure 3.2: GPS Components.**
[Source: Leica Geosystems 2002]

### 3.2.1 The Space Segment

The space segment consists of 24 satellites or SVs (space vehicles). The SVs are in six orbital planes inclined at 55 degrees to the equator at an altitude of about 21,000 kilometres above the surface of the earth. The orbital period is approximately 11 hours and 56 minutes (Leica Geosystems 2002). Each GPS satellite transmits a signal, which has a number of components: two sine waves that are also known as carrier frequencies (L1=1575.42 MHz and L2=1227.6 MHz), two digital codes (the Coarse/Acquisition or C/A and the Precision or P code), and a navigation message. The C/A code is available on the L1 frequency and the P-code is available on both L1 and L2. The C/A-code range measurement is relatively less precise compared with that of the P-code (Young et al.,

2003). It is, however, less complex and is available to all users. The P-code is designed primarily for military purposes and is available only to authorised users (El-Rabbany, 2002). The GPS navigation message contains, along with other information, the coordinates of the GPS satellites, the satellite clock correction, the satellite almanac, and atmospheric data (El-Rabbany, 2002). The codes and the navigation message are added to the carriers as binary bi-phase modulations. The carriers and the codes are used mainly to determine the distance from the user's receiver to the GPS satellites. The transmitted signals are controlled by highly accurate atomic clocks onboard the satellites.

### 3.2.2 The Control Segment

The control segment consists of a master control station, five monitoring stations, and three ground control stations. The Master Control Station (MCS) is located at Schriever Air Force Base, Colorado Springs, Colorado. The primary task of the operational control segment is tracking the GPS satellites in order to determine and predict satellite locations, system integrity, behaviour of the satellite atomic clocks, atmospheric data, the satellite almanac, and other considerations. The MCS collects the tracking data from the monitoring stations and calculates the satellite orbit and clock parameters. These results are then passed to one of the ground control stations for eventual upload to the satellites. The satellite control and system operation is also the responsibility of the MCS.

The monitoring stations are located at Ascension Island, Colorado Springs, Diego Garcia, Hawaii and Kwajelein. The positions (or coordinates) of these monitoring stations are known very precisely. Each monitoring station is equipped with high-quality

GPS receivers and a cesium oscillator for the purpose of continuous tracking of all the GPS satellites in view. All of the monitoring stations and the ground control stations are operated remotely from the MCS. The GPS observations collected at the monitoring stations are transmitted to the MCS for processing. The fresh navigation data are sent to one of the ground control stations to upload it to the GPS satellites through the S-band link. Ground control stations located at Ascension, Diego Garcia, and Kwajalein, are the communication links to the satellites and mainly consist of the ground antennas.

### 3.2.3 The User Segment

The user segment includes all those who use the GPS system in any civilian or military application. With a GPS receiver connected to a GPS antenna, a user can receive the GPS signals, which can be used to determine his or her position anywhere in the world. GPS is currently available to all users worldwide at no direct charge.

## 3.3 Sources of Error

The GPS is affected by a number of systematic errors. The major ones are listed here along with brief descriptions.

### 3.3.1 Signal Degradation (Selective Availability and Anti-Spoofing)

For national security purposes, timing and satellite positional information may be deliberately degraded by the Department of Defence, U.S., through a process of Selective Availability (SA). It is accomplished by manipulating navigation message data and/or the satellite clock frequency. SA primarily affects the accuracy of point positioning using the

C/A code where a single GPS receiver occupies a single point. Using GPS in a relative mode where multiple receivers are used and points whose positions are known are tied into a network probably eliminates most of the effects of SA (El-Rabbany, 2002).

Spoofing is the deliberate transmission of fake signals to misguide a GPS receiver. The Department of Defence has the ability to basically "turn off" the P-Code or to invoke an encrypted Y-Code as a measure of denying access to the P-Code. The purpose is again security to keep opponents from sending false signals with the GPS signature to create confusion. When AS is activated, a special decoding device available only to certain authorized users is required to directly access the P-Code.

### 3.3.2 Timing Error

The GPS satellite clocks (two cesium and two rubidium clocks in each satellite), although highly accurate, are not perfect. Their error has been found to be about 8.64 to 17.28 ns per day (El-Rabbany, 2002). The corresponding range error is 2.59 m to 5.18 m. Thus satellite clock errors cause additional error to the GPS measurements. On the other hand, the GPS receivers use inexpensive crystal clocks, which are much less accurate than that of the GPS satellite clocks. As such, the error caused by receiver clock is much larger than that of satellite clock. It can, however, be removed through differencing between the satellites or it can be treated as an additional unknown parameter in the estimation process. The satellite clock error is also reduced by applying some correction transmitted in the navigation message from the ground control station.

### 3.3.3 Multi-Path Error

Under ideal conditions, a signal would travel directly from a satellite to a receiver. Objects close to the receiver may cause the signal to rebound into the antenna through different paths and affect the range solution. Multi-path is considered to be a major source of error for GPS measurements. Multi-path distorts the original signal through interference with the reflected signals at the GPS antenna. Avoiding positioning the receiver near objects such as buildings, trees or utility poles will aid in the elimination of multi-path effects.

### 3.3.4 Atmospheric Effects (Ionospheric and Tropospheric Errors)

At the uppermost part of the earth's atmosphere, ultraviolet and X-ray radiations coming from the sun interact with the gas molecules and atoms and result in gas ionization. Thus a large number of free "negatively charged" electrons and "positively charged" atoms and molecules are created. Such a region of the atmosphere where gas ionization takes place is called the ionosphere. It extends from an altitude of about 50 km to about 1000 km or even more (El-Rabbany, 2002). The ionosphere, being a dispersive medium, bends the GPS signal and changes its speed as it passes through various layers to reach the receiver. Bending the GPS signal path causes a negligible range error, particularly if the satellite elevation angle is greater than $5^{o}$. It is the change in the speed of signals that causes a significant range error, and therefore should be accounted for.

The troposphere is the electronically neutral atmospheric region that extends up to about 50 km from the surface of the earth. A GPS signal travelling through the atmosphere can be delayed or accelerated by ionospheric and tropospheric effects.

Precise modelling of these effects can prove very challenging. The atmospheric effect can be minimized by keeping base lines relatively short so that atmospheric effects are relatively constant among multiple receivers or by using dual frequency receivers.

### 3.3.5 Antenna Phase Center Variations

The point at which the GPS signal is received by the antenna is called the antenna phase center. Generally, the physical (geometrical) center of the GPS antenna does not coincide exactly with the antenna phase center. It varies depending on the elevation and azimuth of the GPS satellite as well as intensity of the observed signal. As a result additional range error can be expected. By using antennas of the same type and by orienting all receiver antennas to the same direction, typically north, this source of error can be minimized. However, due to its small size (at most a few centimetres), this error is neglected in most of the practical GPS applications (El-Rabbany, 2002).

### 3.3.6 Height of Instrument

GPS is an integrated, three-dimensional system. It is different from lower order classical surveys where horizontal position is determined by observing angles and distances and where elevations are determined by differential levelling in separate and independent operations. Errors in determining the height of the antenna will propagate not only into vertical positions but into horizontal as well. Height from the ground to the antenna center must be measured accurately.

### 3.3.7 Satellite Geometry

The most critical factor in a successful GPS survey is the number and location of available satellites. At least four satellites must be visible for a three dimensional solution. Ideally the four satellites will be well dispersed, one in each quadrant, and at different elevation angles from the receiver's position. Additional satellites are required for some of the newer highly productive observation techniques.

Dilution of Precision (DOP) is a measure of the quality of the satellite geometry. The lower the DOP, the more favourable would be the conditions. Conversely, a higher DOP indicates that the precision is being diluted and the conditions have worsened. DOP values near 4 are considered ideal. If the DOP is above 10, GPS should not be used.

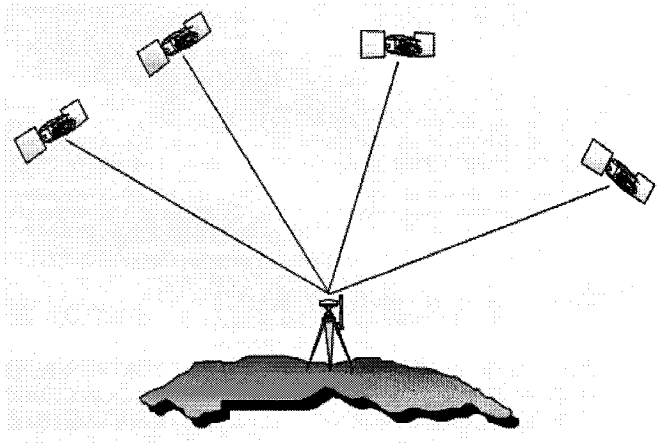
## 3.4 GPS Positioning Methods

### 3.4.1 Single Point versus Relative Positioning

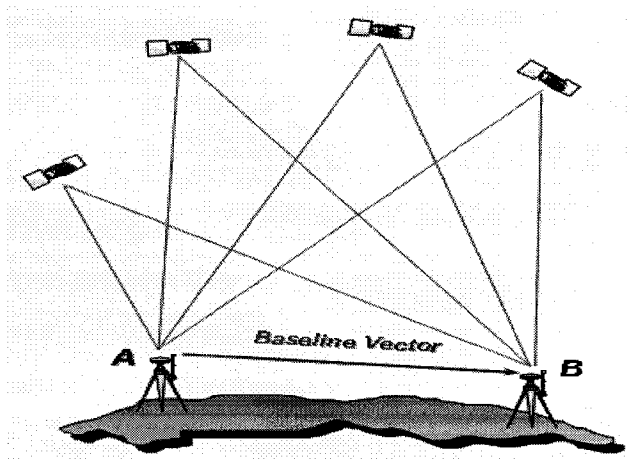
In single point positioning, coordinates of a receiver at an "unknown" point are sought with respect to the earth's reference frame by using the "known" positions of the GPS satellites being tracked (Figure 3.3a). Single point positioning is also referred to as absolute positioning, or often as point positioning. In relative positioning the coordinates of a receiver at an "unknown" point are sought with respect to a receiver at a "known" point (Figure 3.3b). The advantage of using relative rather than single point positioning is that much higher accuracies are achieved because most GPS observation errors are common to the known and unknown sites and are reduced in data processing.

The term "Differential Positioning" is sometimes used interchangeably with relative positioning. However differential positioning is more often associated with a

specific type of relative positioning which applies corrections measured at a "known" site to measurements at an "unknown" site.



**(a) Single Point.**




**(b) Relative Positioning.**

**Figure 3.3: Single Point and Relative Positioning Techniques**

[Source: Leica Geosystems 2002]

### 3.4.2 Static versus Kinematic Positioning

In static positioning, a GPS receiver is required to be stationary whereas in kinematic positioning a receiver collects GPS data while moving. For kinematic relative

positioning one receiver, referred to as a monitor, is left stationary on a known point while a second receiver, referred to as a rover, is moved over the path to be positioned.

### 3.4.3 Real Time versus Post Mission

In real-time processing, positions are computed almost instantaneously, on site. In post-mission processing, data are combined and reduced after all data collection has been completed. Real-time relative positioning requires a data link to transmit corrections from a monitor receiver at a known point to a rover receiver at an unknown point. Post-mission processing for relative positioning requires physically bringing together the data from all receivers after an observation period. Even with real-time point positioning, for many GPS applications it is still necessary to download data and enter it in a database specific to the user's application. Post-mission processing technique gives more accuracy as compared to real-time processing.

## 3.5 Datum

The highly irregular topography of the earth makes the geodetic calculations difficult to determine the location of a point. To overcome this problem, geodesists approximate the irregular shape of earth to a smooth mathematical surface, called the reference surface. For high accuracy GPS positioning, the best mathematical surface to approximate the earth and also keep the calculations as simple as possible was found to be the biaxial ellipsoid. An appropriately positioned reference ellipsoid is known as the geodetic datum. In other words, a geodetic datum is a mathematical surface, with a well-defined origin (center) and orientation. Different countries are using different datums, for

example NAD27 (North American Datum 1927) is used in North America. However, the newer World Geodetic System 1984 (WGS84) is becoming more popular in most parts of the world, even in the United States. As shown in Figure 3.4, it is considered to be the best mean fit to the earth, with origin coinciding with earth's center of mass. In this system, X and Y axes are perpendicular to each other in the equatorial plane, while Z-axis is at right angle to the X, Y plane and coincides with the earth's rotational axis.

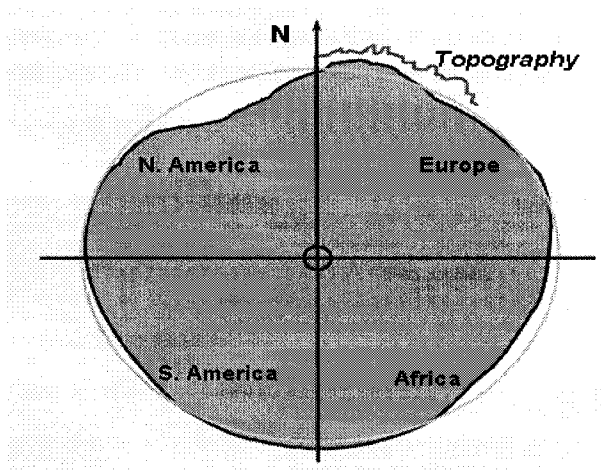



**Figure 3.4: The Best Mean Fit to the Earth, WGS84**

[Source: Leica Geosystems 2002]

## 3.6 Coordinate Systems

The conventional global coordinate system has its origin at the center of the earth, its Z-axis directed towards the North Pole, its X-axis passing through the plane which contains the Greenwich meridian, and its Y-axis perpendicular to the X and Z axes to form a right handed system. Conventional global coordinates are convenient to work with mathematically; however, for many applications they are not as suitable as coordinate systems with more physical significance on earth, such as the geodetic coordinate system.

Geodetic coordinates include the familiar latitude, longitude and height components and are all based on the ellipsoid. Latitude is the positive angle from the center of the earth northward/southward from the equator, longitude is the positive angle eastward/westward from the Greenwich meridian, and the ellipsoidal height is the height above the ellipsoid's surface.

## 3.7 Map Projection

Map projection is defined from the geometrical point of view, as the transformation of the physical features on the curved earth's surface onto a flat surface called a map. It is defined, from the mathematical point of view, as a transformation of geodetic coordinates, obtained from GPS for example, into rectangular grid coordinates often called Easting and Northing. This is known as the direct map projection. The inverse map projection involves the transformation of the grid coordinates into geodetic coordinates. Rectangular grid coordinates are widely used in practice, especially in the geomatics-related works. This is mainly because mathematical computations are performed easily on the mapping plane as compared with the reference surface (i.e., ellipsoid).

Because of the difference between the ellipsoidal shape of the earth and the flat projection surface, the projected features suffer from distortion. In fact, this is similar to trying to flatten the peel of one-half of an orange; we will have to stretch some portions and shrink others, which results in distorting the original shape of the peel. A number of projection types have been developed to minimize map distortions. In most of the GPS applications, conformal map projections are used. With conformal map projection, the

angles on the surface of the ellipsoid are preserved after being projected on the flat projection surface (i.e., the map). However, both area and scale are distorted; areas are either squeezed or stretched. The most popular conformal map projections are Transverse Mercator, Universal Transverse Mercator (UTM), and Lambart conformal conic projections.

It should be pointed out that not only the projection type should accompany the grid coordinates of a point, but also the reference system. This is because the geodetic coordinates of a particular point will vary from one reference system to another.

## 3.8 Geographic Information System

Initially developed in 1960s, GIS is a particular form of information systems applied to geographic data. GIS can be defined as a system of hardware, software, and procedures designed to support the capture, management, manipulation, analysis, modelling, and display of spatially referenced data for solving complex planning and management problems (Yeung 2002). The focus on geographic data and the ability to analyze the data spatially distinguish GIS from other types of information systems. GIS is considered to be one of the most sophisticated software tools for delivering government services, making business decisions, analyzing and presenting spatial information in academic research, as well as helping the general public to understand the world around them.

A GIS is specialized computer software, which shares characteristics with spreadsheets, database, and computer-aided design programs. The distinction of GIS

software is its ability to deal with spatial information in all aspects. It is used as a powerful analytical and decision-making tool. A GIS will include:

- Data input capabilities
- Data manipulation, handling, and analysis capabilities
- Data output capabilities

Geographical data contain four integrated components, namely location, attribute, spatial relationship, and time. GIS is used to explore relationship between features distributed unevenly over space, seeking patterns that may not be apparent without using advanced techniques of query, selection, analysis, and display. With the advancement in the concepts of GIS, its packages are now capable of allowing the users to customize their applications by using a scripting language to build software extensions to meet their specific data-processing and analysis needs. This approach is proprietary in nature, as the scripting language is a software product of a specific vendor and can be used only for developing applications for a particular GIS (Yeung, 2002). It is also possible to build software modules with programming languages, such as Visual Basic, Visual C++, or Power Builder, and to integrate them with the GIS functions originally supplied by the software vendor (Yeung, 2002). In the current study, software was developed in ArcView-GIS using its Avenue programming language.

### 3.9 GPS and GIS in Transportation

The need for more accurate and cost-effective data-gathering techniques, as well as the increasing demand for spatial data, requires that the transportation institutions use the latest and best technology available in the market. One technology that can help make these tasks easier, cheaper, and more accurate is the GPS. This method of determining an accurate position on the face of the earth has revolutionized the way field observations are made. Using satellites, ground-based receiving stations, and computer post-processing, determining a location on the earth's surface has never been more accurate (Czerniak, 1998). Although not intended to measure elevation, GPS can, with post-processing, also provide this type of data. Other effective applications of GPS include using GPS differential positioning to locate, track, and navigate vehicles. The recent emphasis on Intelligent Transportation System (ITS) has put renewed emphasis on the capabilities of GPS for vehicle location and navigation. GPS receivers for these applications are inexpensive compared to geodetic-quality receivers used for surveying.

The introduction of GPS offered new opportunities to transportation researchers for the assessment of driving behaviour on highways. However, until the year 2000, the accuracy of positional data through GPS was being reduced by U.S. Department of Defence, for the non-U.S. military users (Porter et al., 2004). As mentioned earlier, this option was possible through the intentional degradation of GPS signals, referred to as SA. This SA leads to positional errors of up to 100 meters. In order to get more accurate data (within 1 meter of the actual position) with SA on, either real-time or post-processing correction (differential correction) of the data is required (Porter et al., 2004). On May

01, 2000, SA was turned off by the U.S. government, so that any GPS receiver could have errors of 20 meters or less, without making any correction (Porter et al., 2004).

In order to assess the difference between differentially corrected and uncorrected GPS data derived during driving in an SA off situation, a study was carried out by Porter et al. (2004). The purpose of the study was to determine the need of differential correction for the assessment of driving. Only three variables of the driving behaviour were considered, that is vehicle position, velocity and acceleration. Results of the study indicated that there are no significant differences between any of the variable pairs when comparing corrected to uncorrected data, except for the position measurements (10% difference observed). While comparing the Easting and Northing position data, more error was observed in the uncorrected Northing direction.

A GPS survey can be used to track the existing highway alignments. However, the GPS data need to be consistent, otherwise it would need some treatment. A methodology was developed by Ben-Arieh et al. (2004) to post process GPS data. They used a huge amount of GPS data, repeated for the same highway several times with a noticeable inconsistency. The inconsistency was attributed to the vehicle driving in both directions (there was as much as 60 meter distance between lanes), varying driving pattern of the vehicle (changing lane, getting off the road for refuelling, etc.), seasonal bias and system inherent inaccuracy. In the first step, the geodetic coordinates were converted to a Cartesian system. Secondly, the data were sorted along one major direction of the highway. Next, the data were cleaned of repetition and inaccuracy using an algorithm developed for the same purpose. Finally an accurate geometric representation of the highway was generated using B-Spline approximation. The

observed alignment showed high accuracy when compared with design data of the same highway. Work is in progress to quantify the deviation of the observed alignment from its designed position.

Keeping in view the spatial attributes of transportation data, the Transport System Centre (TCS), South Australia, has developed an integrated GPS-GIS for collecting on-road traffic data from a probe vehicle (Taylor et al., 2000). They used GIS as a database management platform for the integration, display and analysis of the data collected from GPS and the in-vehicle instrumentation. The system proved to be reliable and pertinent because of:

- The ability to obtain second-by-second position data and simultaneous speed profile data from the GIS;
- The spatial display and storage of data in GIS environment for interpolation, analysis, and integration with other data sets; and
- The ease of transferability of GPS equipment from vehicle to vehicle.

It is also a fact that each group of transportation researchers uses its own GPS equipment, GIS database, and internal set of GPS data processing rules, and that information sharing and coordination has been limited (Czerniak et al., 2002). Hence, keeping this fact in view, a project was taken by the National Cooperative Highway Research Program (NCHRP) to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject area of interest. The objective of the synthesis was to specify the major issues associated

with the combined use of GPS and GIS and how to address them for digital mapping applications related to transportation. In this report (Czerniak et al., 2002), two main sources of information were used; literature relating to transportation, GPS, and GIS, and the collective experience of transportation departments, and consultants, using a survey questionnaire. It was found that using GPS and GIS for mapping has increased accuracy, decreased costs, reduced project completion time, and improved overall map quality. At the same time, the integration of GIS and GPS has created new problems, including identification of inaccurate and bad data points, missing data points, and the lack of standard map matching algorithms, that must be overcome if the full benefit of the two technologies is to be obtained.

The most serious integration problem is spatial mismatch or map matching. Spatial mismatch can be defined as the lack of similarity between a feature on the earth's surface as specified by a GPS point and the location of the same feature within a digital base map. In other words, the two features do not line up. This can occur because of poor GPS data collection procedures, limits to the basic accuracy of the GPS unit, limit to the accuracy of the GIS source data, a flawed GIS digital base map, or some combination thereof. Two other reasons for spatial mismatch are the use of different map projections and different accuracy specifications.

Another potential area for GPS-GIS application in transportation engineering is collection and analysis of collision data. The current practice of crash data collection through police reports has been reported to be time consuming and inaccurate (Pisano et al., 1996). Errors which occur during various steps of data collection diminish the data's value since the data do not necessarily reflect the actual circumstances of the crash. These

errors propagate through the system, affecting the quality of the safety analysis, as well as the value of any decisions based on these analyses. To overcome the problem, the Federal Highway Administration (FHWA) has set up a program that includes the applied research of advanced technologies to improve highway safety. The main focus of this program is to improve the methods to collect and analyze highway safety data, primarily through the application of technology, including hardware (computers, GPS, image based measurement tools) and software (GIS, Statistical Analysis System, etc.).

# Chapter 4

# METHODOLOGY AND DATA COLLECTION

This chapter provides a description of the data collection methodology adopted for the study. It includes details about the techniques and instruments used for the survey, the criteria for selection of sites and segments of highways studied, sources of data, and a list of variables to be analyzed in the study. Although GPS positioning techniques vary significantly as discussed in Chapter 3, their procedures for data collection may be grouped into three common phases: planning and preparation, field operations and data processing.

## 4.1 Planning and Preparation

Planning and preparation for a GPS field survey begins with the identification of positioning requirements and ends with complete readiness for successful field operations. The extent of all the intermediate steps varies greatly with the magnitude, accuracy, and locality of the projects. Important steps within the planning and preparation phase include site selection, selection of positioning technique, selection of equipment, validation, reconnaissance, survey design and preparations. Many of these planning steps are quite interdependent.

As a preliminary step, the highways to be tracked and the nearby available control stations, were selected. Initially, data for seven highways namely Highways 7, 12, 15, 17, 31, 41 and 43; were available for the proposed work (Table 4.1). But since control stations were available only for Highways 41 and 31, the data collection was limited to

these two highways. These highways were selected primarily because of data availability; however, they do contain features representative of most two-lane rural highways found in Ontario. The selected segment of Highway 31 passes through a mostly flat area, whereas that of Highway 41 constitutes a mostly hilly terrain (Figures 4.1 and 4.2).

**Table 4.1: Summary of Selected Highways**

| Highway Number | Highway Classification | Control Stations | Segment length (km) | Total number of curves |
|---|---|---|---|---|
| 43 | Secondary Highway (Former King's Hwy) | Unavailable | 17.2 | 7 |
| 41 | King's Highway | Available | 26.5 | 16 |
| 31 | Secondary Highway (Former King's Hwy) | Available | 33.5 | 12 |
| 17 | King's Highway | Unavailable | 54.5 | 13 |
| 15 | King's Highway | Unavailable | 16.0 | 12 |
| 12 | County Road | Unavailable | 31.4 | 18 |
| 7 | King's Highway | Unavailable | 13.5 | 11 |

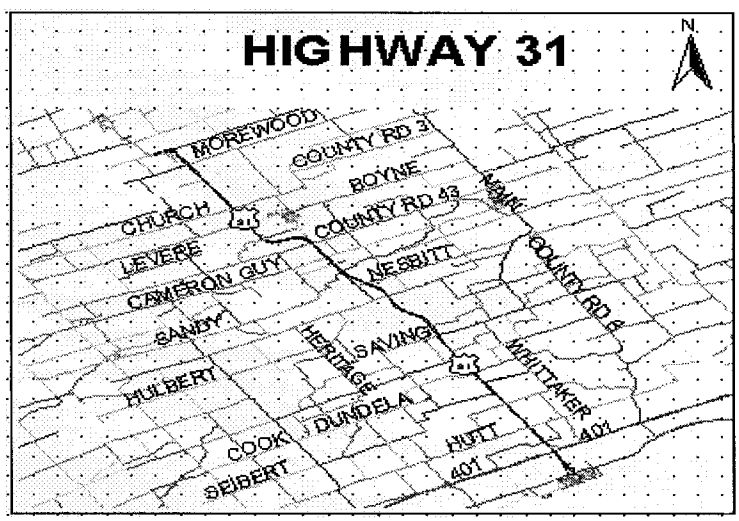



**Figure 4.1: Digitized Map Showing Selected Section of Highway 31.**

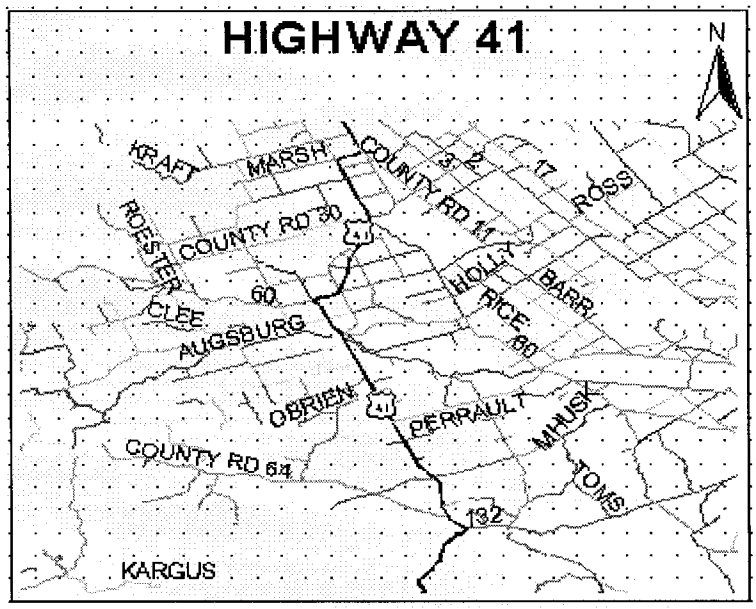
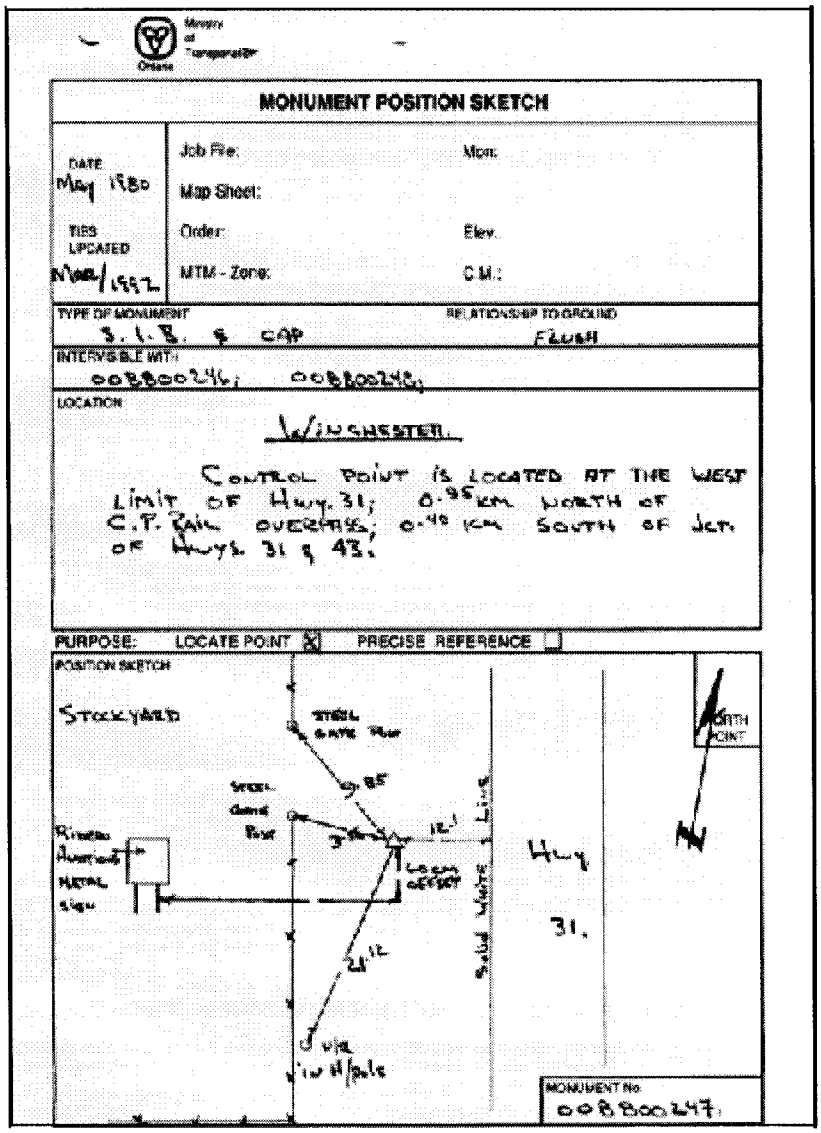



**Figure 4.2: Digitized Map Showing Selected Section of Highway 41.**

Three types of data were used in this study. These data and their respective sources are:

1. Geometric alignment data for the selected test sections were extracted from the as-built drawings provided by the Ministry of Transportation (MTO) and concerned County Office. The planning and design office for the Eastern region of the Ontario Ministry of Transportation (MTO) provided data for the King's Highway 41, while the roads department of the United Counties of Stormont Dundas and Glengarry (SDG) provided data for the Secondary Highway 31.
2. Data about GPS control stations including their location plans and coordinates data were obtained from the Geomatics Office of the Ministry of Transportation, Ontario (Figure 4.3).

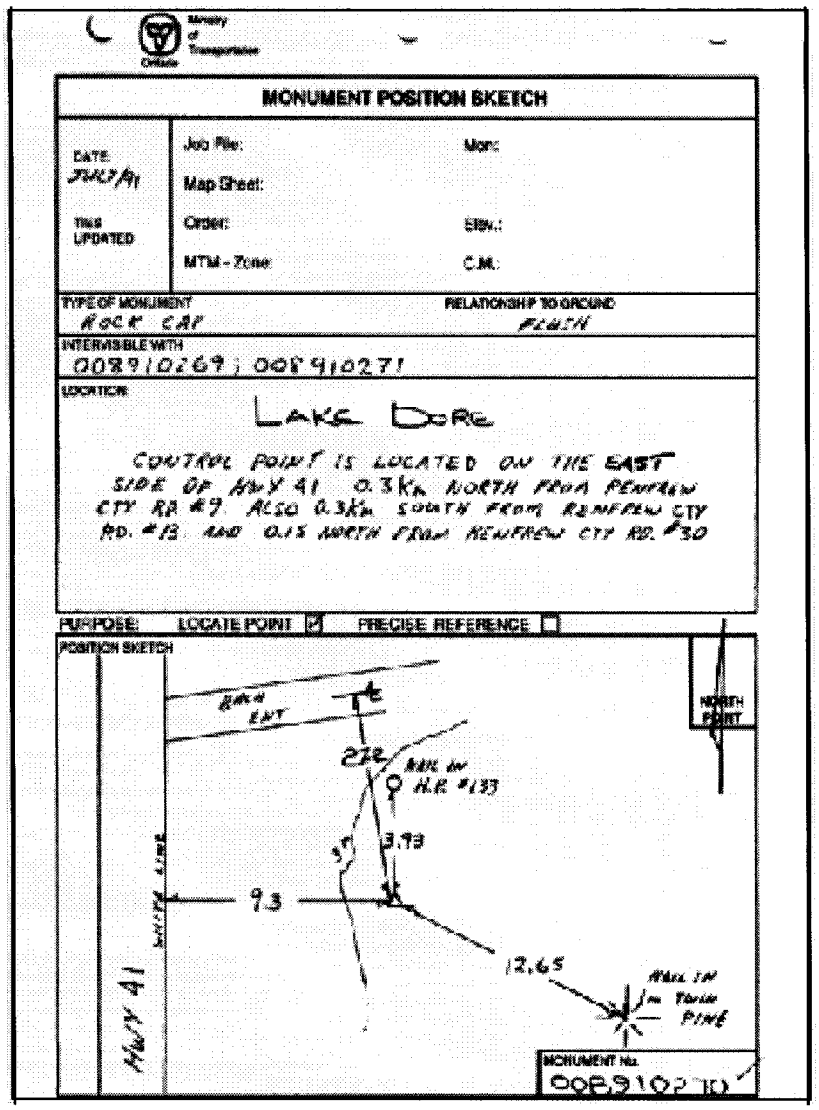
3. Observed coordinates data of vehicle travelling path were collected using the "Differential GPS" survey technique.




**(a) Highway 31.**

**Figure 4.3: Location Plans for Control Station on Highways 31 and 41.**

[Source: Geomatics Division office, Ministry of Transportation, Ontario]




**(b) Highway 41**

**Figure 4.3: Location Plans for Control Station on Highways 31 and 41 (Cont'd).**

[Source: Geomatics Division office, Ministry of Transportation, Ontario]

The highway numbers, their classifications, segment lengths and number of horizontal curves available are summarized in Table 4.1. A combined length of 60 km on Highways 31 and 41 with 28 horizontal curves was initially available for analysis. As explained later, the final number of highway curves investigated in this study was reduced to 17 with varying characteristics.

A summary of the main data variables regarding alignment that were collected from the as-built drawings is:

- Radius of curve.
- Length of curve.
- Length of spiral (if present).

### 4.1.1 Site Selection

A final sample of 17 curves was selected from the total number of 28 curves available in the two highway segments under study. The curve selection criteria were based on the following constraints:

1. Rural area; not near town or built up area.
2. No high-rise buildings, tall trees, overpasses, and other obstructions which may cause interference to the signal.
3. Availability of control station (for differential GPS survey).
4. Relatively low traffic volume: AADT $\leq$ 10,000 veh/day.
5. Marked and paved roadways with constant lane width for easy tracking of the centerline.
6. No stop control or signalized intersections within 0.8 km of curve for driving at a constant speed.
7. Curve radius < 1000 m, and curve length > 50 m

These criteria generally attempt to eliminate features that may hinder an accurate data collection, and are mostly based on an understanding of the impact of different roadway features on safety. Moreover, since the data were planned to be collected at constant speed, the above-mentioned criteria help in achieving and maintaining a desired speed.

The highway plans were first reviewed to identify the full list of all horizontal curves available in each highway segment, which are summarized in Table 4.1. These curves were then closely inspected to establish potential sites based on the site selection criteria above. This process resulted in 17 potential curves for data collection and analysis. Since the plans were relatively old, it was expected that there would be a considerable number of changes in these areas. A major concern was the likelihood of many of these rural areas turning into small towns with time. Thus, the sample selection was not finalized before a windshield survey was conducted to review the actual road conditions and nearby land features, particularly new construction, intersections, and the posted speed limit. Ultimately, several curve segments were eliminated as a result of these site visits due to the presence of buildings and signalized intersections that were not reflected in the plans.

The actual geometric alignment data for the selected highway segments were extracted from the as-built construction plans, and stored in a spread sheet (Tables 4.2 and 4.3). The data obtained from the plans included stations of the point of intersection (PI), curve radius, curve length, and length of spiral if present. Both Highways 31 and 41 have the same 80 km/h speed limit.

**Table 4.2: Geometric Features of Selected Curves on Highway 31**

| Curve | Station (PI) | Radius (m) | Length (m) | Spiral (m) |
|---|---|---|---|---|
| 1 | 583+91.94 | 873.20 | 464.20 | 61.00 |
| 2 | 621+53.91 | 873.20 | 474.00 | 61.00 |
| 3 | 12+104.34 | 436.60 | 162.40 | 61.00 |
| 4 | 12+572.90 | 580.00 | 170.00 | 61.00 |
| 5 | 15+931.38 | 589.00 | 341.25 | 61.00 |
| 6 | 16+551.18 | 583.00 | 377.33 | 61.00 |
| 7 | 17+978.44 | 583.00 | 783.64 | 61.00 |
| 8 | 26+621.13 | 349.00 | 472.38 | 61.00 |
| 9 | 27+731.04 | 388.00 | 527.00 | 61.00 |

**Table 4.3: Geometric Features of Selected Curves on Highway 41**

| Curve | Station (PI) | Radius (m) | Length (m) | Spiral (m) |
|---|---|---|---|---|
| 1 | 126+90.93 | 388.00 | 159.79 | 61.00 |
| 2 | 136+93.09 | 582.12 | 110.91 | 61.00 |
| 3 | 205+35.50 | 873.19 | 430.66 | 45.73 |
| 4 | 256+68.03 | 873.19 | 208.79 | 45.73 |
| 5 | 336+30.65 | 806.00 | 541.13 | 45.73 |
| 6 | 356+86.57 | 582.12 | 71.12 | 45.73 |
| 7 | 382+00.02 | 436.59 | 100.14 | 45.73 |
| 8 | 397+06.63 | 436.59 | 204.66 | 45.73 |

### 4.1.2 Selection of Positioning Technique

There are many aspects which influence the choice of positioning technique, such as accuracy requirements, geographical environment, distance between points to be positioned and the costs. The cost of GPS positioning is closely tied to the technique used, which in turn is chiefly a product of the accuracy requirements. Two major reasons for cost variations with technique are the time required on site and the cost of the required receivers. Generally the shorter the time required on site, the lower the survey cost.

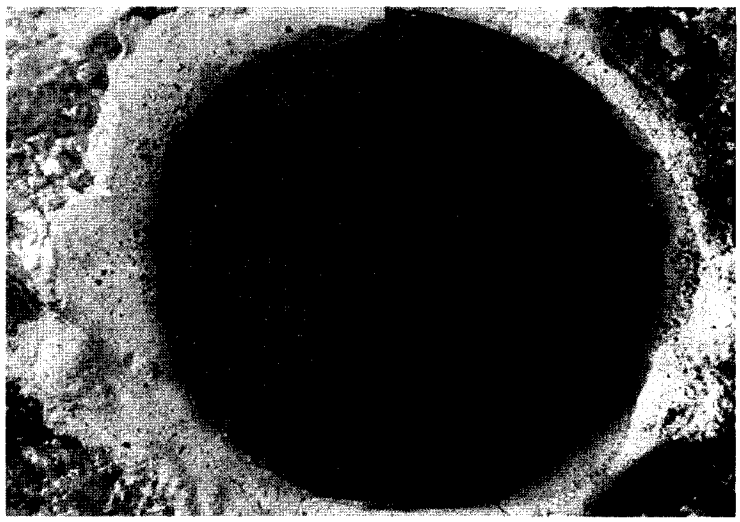
Of various techniques discussed in Chapter 3, the Differential GPS survey technique was adopted to obtain maximum possible accuracy in the positioning data. This technique is good from accuracy point of view but is relatively expensive, because of the following reasons:

- Two sets of all equipment (GPS receivers, antenna, tripod etc) are required; one for the control station and another for the rover.
- A point with known coordinates (control station; shown in Figure 4.4-a) is needed; which if not readily available would be established by static survey technique with a minimum of 72 hours continuous tracking of GPS satellite signal.
- Extra time is needed for locating the control stations, centering and levelling the tripod over the central point of the control station, particularly when it is located on rocks (as was the case on Highway 41).

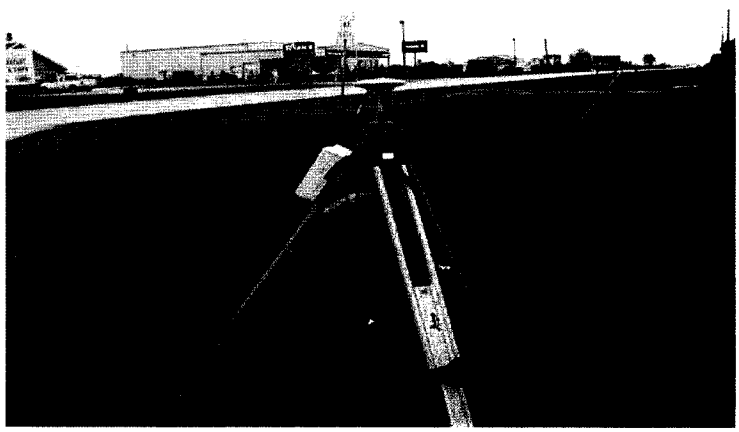
It is advised in the GPS Positioning Guide (1995) that all receivers used together for relative positioning should be of the same make to avoid problems which often result from mixing receiver types such as biases, complexities in data processing and data rate incompatibilities. Accordingly two GPS units of the same type (SR 520) by Leica Geosystems were used in the survey. One unit was fixed on a tripod at the control station nearest to the highway; while the other unit was placed atop the test vehicle, to work as the rover station. An antenna of type "AT 502" was used to receive the satellite signals. For the control station, the antenna was fixed on the top of tripod (Figure 4.4-b); whereas for the rover station, antenna was fixed on a magnetic mount atop the car (Figure 4.4-c).

### 4.1.3 GPS System 500

A system 500 unit was used to receive signals from GPS satellites, which were then processed to obtain a position on the earth's surface. Its main components are the GPS antenna and GPS receiver (Figure 4.4). Ancillary components include a terminal, batteries with charger, PC cards and cables. A computer-based software namely "SKI-Pro" is also used in conjunction with the aforementioned hardware for post-processing GPS data and for downloading coordinates recorded in the field.



**(a) Control Station on Highway 41.**



**(b) GPS Set Up at Control Station on Highway 31.**

**(c) GPS Set Up as Rover Station.**

**Figure 4.4: Control Station and Setup of GPS Equipment.**

Of the six available models of system 500, GPS system SR 520 was used for the current study. This system tracks the L1 C/A code and L2 P-code to reconstruct the carrier phase. When AS is activated, the receiver switches to a patented P-code aided tracking technique that provides full L2 carrier measurements and L2 pseudoranges. Data can be stored for post-processing. Baselines can be calculated with a precision of up to about 3-10 mm + 1 ppm. This receiver can also be used for real-time measurements with the attachment of a radio modem.

Dual frequency antenna AT502 was attached to the GPS receiver through a cable. The antenna receives the GPS signals which are then processed by the receiver. The TR500 terminal was attached to the receiver to provide a full user interface to the system. It is used for configuration of the receiver as per job requirement and for steering the GPS operation. Data input is made through a fully alphanumeric QWERTY keyboard and an LCD display of 32×12 characters which may be illuminated.

Field data were stored on a PC card of 16 MB capacity inserted into the slot on front of the GPS receiver. After field survey this PC card was inserted into a laptop computer and data were downloaded and saved in the proper database. The PC card was then formatted for next time use. Two separate PC cards of the same type were used, one in the control unit and other in the rover unit. Two fully charged GEB 121 camcorder type batteries were used to be plugged into the underside of the GPS receiver to power the system (receiver and terminal) for about six hours continuously.

### 4.1.4 Validation

In the planning phase of a GPS project the procedures and equipment to be used from data collection to the final product, should be tested to ensure that they reliably satisfy the desired accuracy requirements. This testing is referred to as the validation process. If a user has previously successfully employed the same GPS procedures and equipment for a similar application, revalidation may not be necessary.

Three main components are tested in the validation process; the positioning technique chosen, the equipment to be used and the processing method adopted. It cannot be assumed that all accuracy claims given by the equipment manufacturers or other users will be consistently met under all field conditions, and hence it is important to test and evaluate the equipment. The validation process also has the benefits of enabling users to identify and solve problems before commencing costly field surveys, to streamline operations. Validation testing was carried out within the Carleton University Campus.

Validation is an important evaluation and feedback step in the project plan. An equally important step which gives feedback in the planning process is field reconnaissance.

### 4.1.5 Reconnaissance

Reconnaissance consists of checking the field project sites before commencing GPS observations. Sites should be checked for their suitability for GPS survey, availability of control stations, and logistical requirements

A good GPS site should be free from obstructions and interference. Through field reconnaissance, obstructions or interference may be identified and avoided by alternate

site selection. As discussed in Chapter 3, obstructions are obstacles (buildings, trees etc) which block the line of sight between a satellite and a receiver, thereby preventing signal reception and causing multipath errors. To avoid satellite blockage, ideally a site should be obstruction-free in all directions above a 15 degree elevation angle (measured from horizontal line at antenna level). In less than ideal conditions where some obstructions do exist, successful positioning may be possible if a sufficient number of satellites with adequate geometry can still be tracked. For the current study, Highway 31 was situated in an ideal environment whereas Highway 41 was found to be in less than ideal conditions.

During field reconnaissance, control stations planned for use were checked for availability and suitability for GPS observations. The final product of field reconnaissance included a set of points ready for GPS observations as well as a current description for each site, access information and a description of any special steps which need to be taken.

## 4.2 Field Operation

With good planning and preparation, field operation should be relatively smooth. During field operation, responsibilities were distributed amongst the members of the survey team. The number of team members depends upon the type of survey and the amount of work involved. For purely kinematic GPS survey only one person is needed for taking observations in addition to the car driver. In differential GPS survey, a third person may be needed to stay at the control station for theft prevention and protection from any interference. Before departure for the survey, the equipment was checked and the receiver batteries were fully charged. The two PC cards (memory devices) were

formatted to provide full storage capacity. Keeping in mind the location of the target highways, sufficient time was planned for the travel as well as the survey work. For this purpose, the usual departure time from Carleton University was 7:00 AM.

### 4.2.1 Setting and Configuration of Control Station

After reaching the site, the control station was located. The control unit was set up over the control station as shown in Figure 4.4, according to the procedure mentioned below:

- The tripod was set and centered on the control station marker/monument.
- The tribrach was mounted and levelled on the tripod. Tribrach was used for precise levelling and centering of the antenna.
- The carrier to hold the antenna was placed, locked in the tribrach and centered over the intersection of cross-hairs engraved in the survey marker/monument using optical plum bob.
- The antenna was screwed onto the carrier.
- The tribrach was checked to ensure it was still levelled. During observations it was regularly checked for precise centering and levelling.
- The height hook was inserted into the carrier to measure the height of antenna from the antenna phase center to the survey marker, and then removed from the set up.
- The GPS receiver was connected to the antenna using the GEV120 antenna cable.

- The GEB121 batteries were plugged into the GPS receiver.
- The TR500 terminal was attached to the receiver.
- The PC card was inserted into the receiver.
- The system was switched on using the ON/OFF button in the terminal.

The system was then configured to work as a base-station following the instructions provided in the manual of the GPS system (Leica Geosystems, 2002). Accordingly, in the configuration menu the antenna type was selected as "AT502 Tripod" and its height was entered as per measurement. The coordinate system was selected as decimal degrees and the observation rate was entered as 0.1 second, i.e., 10 observations in one second. To facilitate file management a unique job-name was given to each session of the survey. As the data observed at base station are usable for each session of rover data being observed simultaneously within the same time, only one job name was entered for the base station. After pressing the "OCCUPY" key, the receiver started recording static observations until the "STOP" key was pressed.

### 4.2.2 Setting and Configuration of Rover Station

The equipment set up for the rover station, as shown in Figure 4.4, was made according to the following steps:

- A magnetic mount was set on top of the car very carefully so as to avoid scratches to body of the car, as the powerful magnet was attracted to the car

body very quickly. A layer of cloth was used between the mount and car surface to avoid scratches while removing the mount.

- The antenna was screwed into the mount in such away that its cable connection lead was towards the car window side.
- After connecting the terminal TR500 to the GPS receiver, the PC card was inserted into the receiver and the GEB 121 batteries were plugged into the receiver.
- The GPS receiver was connected to the antenna using antenna cable passing through the car window.
- The system was switched on using the ON/OFF button in the terminal.

The rover unit was then configured for kinematic GPS survey. Antenna height was measured from top of pavement to the center of antenna already fixed on the car top. This measurement was entered to the receiver after selecting type of the antenna as "AT502 pole". By default the antenna height for type "AT502 Pole" was 2.0 meters; however, using Edit key in the terminal, this value was changed to the measured value. Coordinate system and the observation rate was selected the same as for the base station. Different job name was selected for each of the six runs of rover unit at each site as explained later. The option for logging "moving observations" was checked to be yes. The measure of quality of position was selected as "DOP" (Dilution of Precision) as defined in the manual (Leica Geosystems, 2002). For SR520 GPS receiver, the static initialization was set to "OFF". After completing the configuration,

observations were started as soon as the "CONT" key was pressed in the "SURVEY" screen.

After recording observations for a couple of minutes in static position the car was accelerated to the required speed. The driver was instructed to drive in the middle of the lane as much as possible. During travel, status of observations was regularly checked using the "STATUS" key available in the terminal. Speed of the car was also displayed during travel; however it could not be recorded in the post-processed survey technique, and could only be recorded in real-time survey. In Highway 41, at a couple of spots, the DOP value exceeded its limit and the receiver gave a warning sound indicating loss of lock with the satellites. However, after a short time it stopped warning as soon as the DOP value came down within the limit. After reaching the last point the survey was stopped and the observations were stored on the PC card.

Three runs with rover unit were made in each direction of the highway. During the first run the car was driven at the speed of 80 km/h in both directions; the second run was at 90 km/h and the third run was at 100 km/h. It should be noted that the choice of these speeds was based on the results of a speed study conducted earlier on the same highways (Misaghi, 2003). When all field work was completed, the equipment was disconnected and the batteries and the PC cards were taken out of the GPS receivers. During field work, important notes about the site, weather, timings, etc., were taken which were later compiled into report shape. Data from both PC cards were downloaded and stored into a personal computer using the relevant software of the GPS, namely SKI-Pro. The known coordinates of the base station were entered and the raw GPS data were processed for removal of the ambiguities.

## 4.3 Data Post-Processing

### 4.3.1 SKI-Pro, the Software

The data collected in the field were in raw form and contained ambiguities. To resolve these ambiguities and acquire refined data in our desired coordinate system, the GPS raw data were imported into the SKI-Pro (Static-KInematic-Professional) software (version 3.0) and post-processed. SKI-Pro is a complete GPS office software package that accompanies the Leica GPS System 500 hardware, used for data collection. It is a comprehensive set of programs for GPS surveying including post-processing and real time measurements. Its various functions include:

- Data importing
- Data management
- Data processing
- Network adjustment
- Datum transformation
- Data exporting

### 4.3.2 Data Post-Processing

As mentioned earlier, during GPS survey, data were stored on PC card, which were later saved in a computer database. After activating SKI-Pro, a new project was created for importing the GPS raw data. Using the "Import raw data" option, all the GPS raw data in the selected folder/file were imported in one run. In kinematic survey, raw

data are first imported for the Control/Base station GPS unit and then for the Rover GPS unit in the same project.

In the project window various options are available to display the contents of the project by using different views. As shown in Figure 4.5, different views can be displayed by clicking the tabs available at the bottom of the window. Selecting "View/Edit" tab shows a graphical representation of each point in the project. A point represented by the symbol "+" indicates that this point is still navigated (in raw form) and its accuracy is low (±100 m). In "Properties" option of the point representing a Base Station, the point class is changed to "control" and is indicated by the symbol "▲". Also, the coordinates of the selected Base Station are changed to the known values, which in this study were obtained from MTO. The "GPS-processing" window is then opened by clicking its respective tab at the bottom of the screen. As shown in Figure 4.6, the new view displays a list of all observation intervals and a graphical representation of the observation time for each interval. In the graphical window all observations are represented by horizontal bars which can be selected as Rover (color will change to green) or reference (color will change to red). After selecting the Base Station and the Rover points the raw data are processed by choosing the "Process" option in the menu.

After completion of the processing run, the "Results-View" is automatically displayed, allowing one to examine and store the processed data. All Rover points are listed together with their coordinates, quality and ambiguity status. The points for which the ambiguity status is yes are selected automatically. Ambiguity status "Yes" indicates that the baseline calculation is correct i.e., ambiguities are resolved. Ambiguity status "NO" indicates that the ambiguities could not be resolved. Ambiguity status "?" indicates

that no attempt has been made to resolve the ambiguities. By default ambiguities can only be resolved for baselines up to 80 km. For longer distances the ambiguity resolution becomes unreliable. In the current study, the control stations were situated along the edge of selected highways, and the length of the baseline was within the required limit.

The selected post-processed points are stored in the database by choosing the "Store" option provided in the pop-up window. One can see the graphical representation of the processed points by clicking the "View/Edit" tab. Selecting "chain" in the graphical setting connects all the points in the proper directions and show the trajectory line as shown in Figure 4.5. In Figure 4.5, point "A000001" is the first occupied point and the circle around it indicates that its related ambiguities have been resolved. Points without resolution of their ambiguities are represented by squares. The triangular "Base" point indicates control station. The "New" point at the other end is just to show that additional points can be added to the database in addition to the navigated points. Location of a new point is selected graphically using the mouse, hence the accuracy of its coordinates depends on the resolution of the monitor screen as well as on the size of the area being displayed (zoom status).

The resulting data can be operated using various options available in the software. Any undesired data columns can be hid in the results table and thus the results can be arranged in the desired parameters. The coordinates can be converted to local grid system (Easting-Northing-height) from the default Cartesian coordinate (X-Y-Z) system. Data can be exported in text format and can be worked upon in other packages like word pad, Excel and Geographic Information System. A report summarising all details about the project is also available in the results (Figure 4.7).

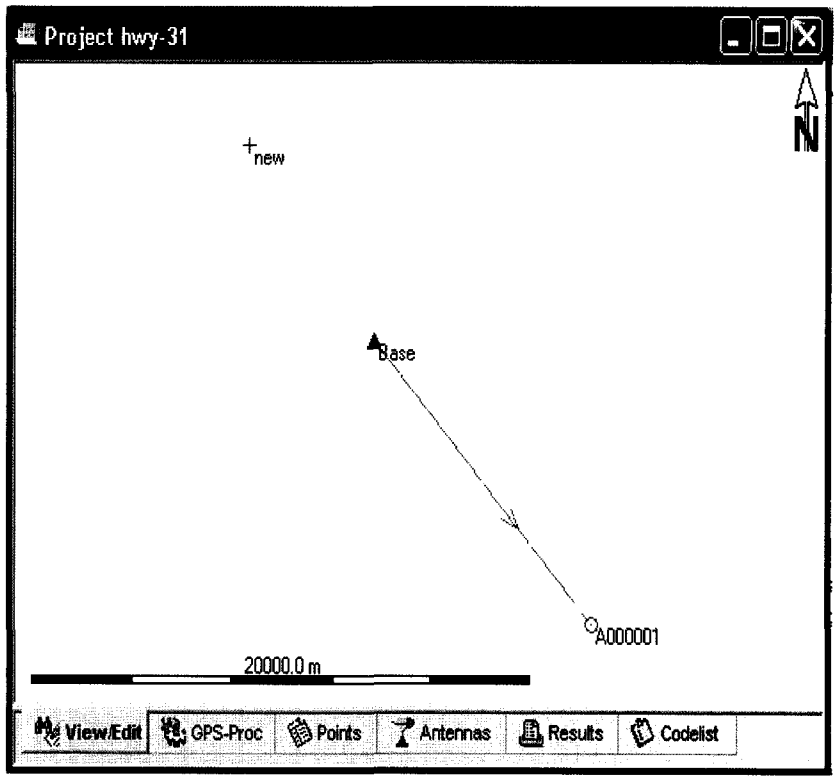



**Figure 4.5: Project View in SKI Pro, after Post-Processing of the GPS Data.**

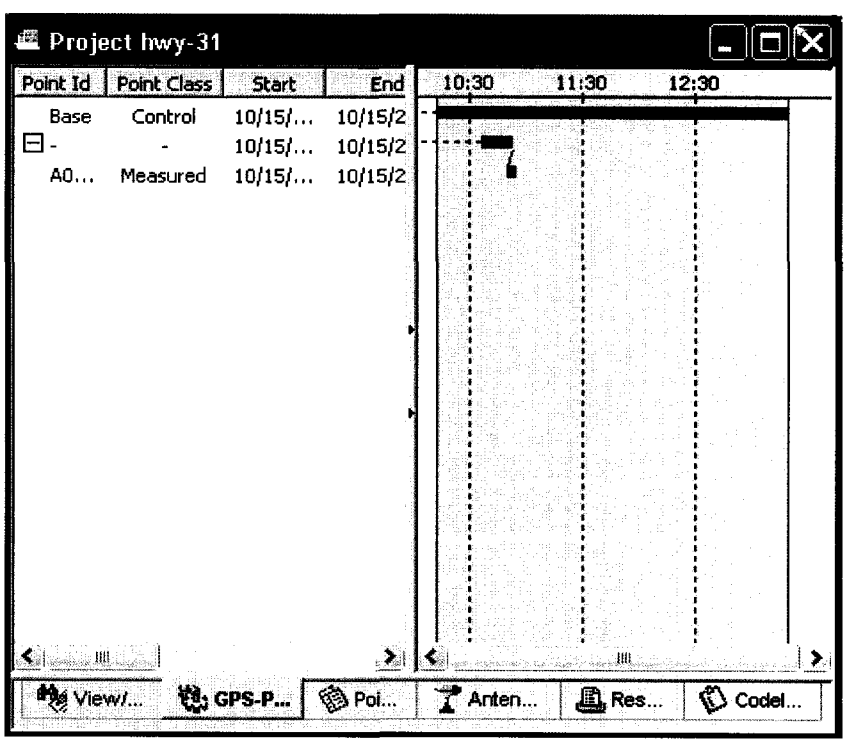



**Figure 4.6: GPS Processing View, with Selected Rover and Control Points.**

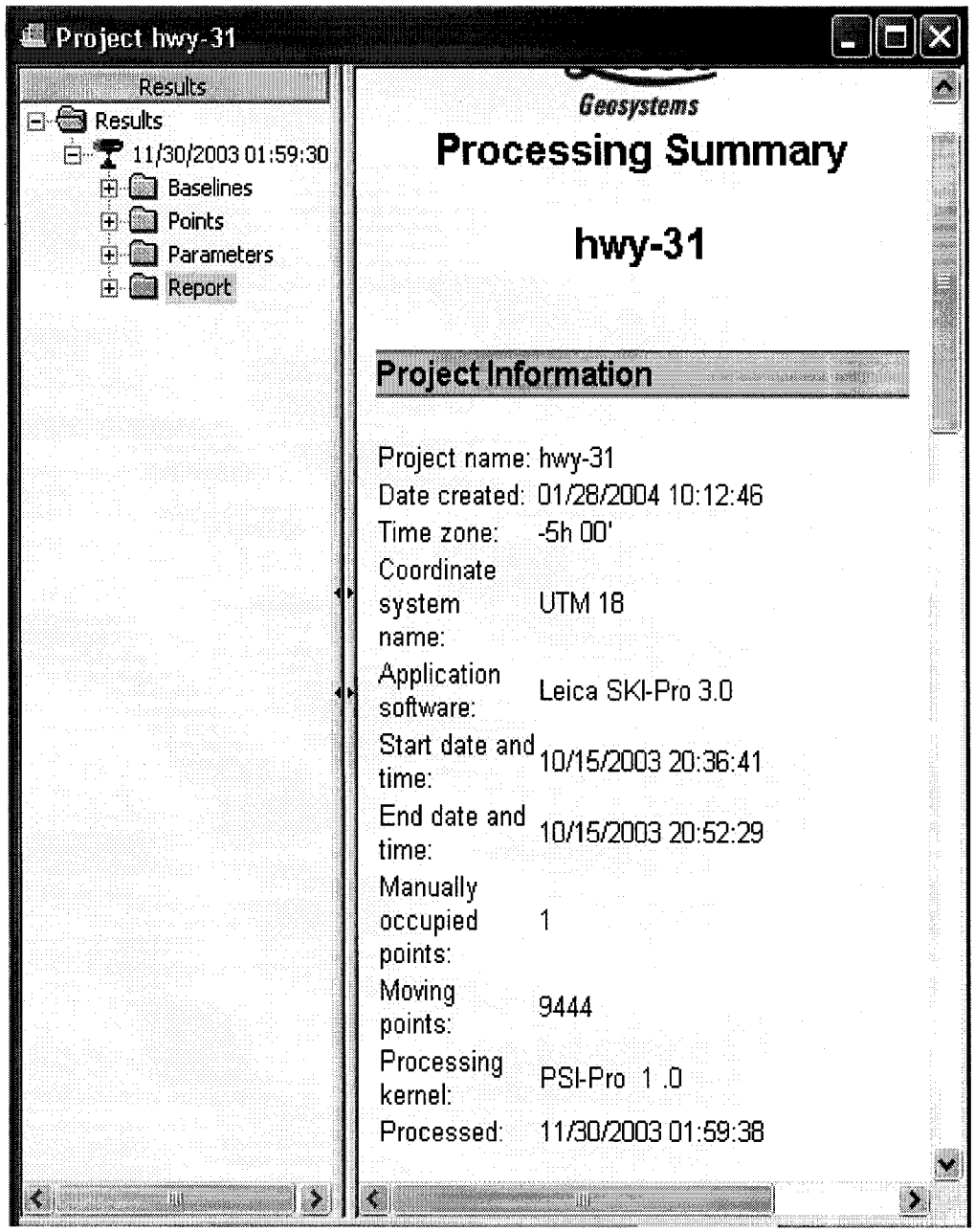



**Figure 4.7: Project Report Summarising All Information about the Processed Data.**

# Chapter 5

## FITTING THE HORIZONTAL ALIGNMENT

### 5.1 Overview

The collected GPS data appear as points (point shape) when displayed in a GIS environment. Due to slight movements/vibrations during driving, the real data points were not collinear. In order to fit highway horizontal alignment into the real data and determine the necessary geometric features of horizontal alignment a computer program was developed as an ArcView's extension using its programming language "Avenue". Avenue and the Dialog Designer Extension were used to customize and develop the application. The development efforts for the new extension, called "Roadfit" are presented in this chapter.

### 5.2 Extension Development

Extensions perform new functions within ArcView that were not available previously. Generally, extension program code can be saved as part of an ArcView project or can be written out to a text file. Extensions consist of scripts and may or may not have a graphical user interface (GUI).

The GPS data were converted to point shapefiles, which is ArcView's native file format. The extension is designed to analyze segments containing simple horizontal curve with or without spiral transitions and tangents. The algorithm for the developed extension was finalised after going through a step-wise procedure including use of mathematical, statistical and geometrical relationships as explained below:

### 5.2.1 Fitting Algorithm

Three components of the actual alignment of a highway were considered in the extension development: namely tangents, spiral curves, and circular curves. The approach involved four basic steps: separate points related to the straight and curve segments, fit straight lines to tangent sections, fit circular curves, and join the remaining points as transition curves.

#### 5.2.1a Separation of Points

Points were initially separated into categories by comparing the angles of consecutive segments using the equation:

$$\theta_i = \tan^{-1}\left(\frac{y_{i+1} - y_i}{x_{i+1} - x_i}\right); \text{ for } i = 1, 2, 3, \ldots \tag{5.1}$$

Where $x$ and $y$ = the coordinates of any two consecutive points.

If $\theta_i$ is equal to $\theta_{i+1}$ within some tolerance, $\delta$ degrees (that is $|\theta_i - \theta_{i+1}| \leq \delta$), the points are considered to lie either on the straight entry segment or on the straight exit segment with the remainder belonging to curved segment(s) of the highway. Thus the two types of data points are separated and kept in two different pools. These two pools of points were then fitted to straight lines and circular curves, respectively.

#### 5.2.1b Fitting Straight Line

The general form of linear equation for fitting a straight line is

$$y = ax + b \tag{5.2}$$

Where $a$ and $b$ = constants determined using linear regression.

The most common method for linear regression is the method of least squares, in which the values of the constants $a$ and $b$ are determined so as to minimize the sum of the squared deviations between the original data (observed coordinates) and that predicted by Equation 5.2. The values of $a$ and $b$ are given by:

$$a = \frac{n\sum x_i y_i - \sum x_i \sum y_i}{n\sum x_i^2 - (\sum x_i)^2} \tag{5.3}$$

$$b = \frac{1}{n}(\sum y_i - a\sum x_i) \tag{5.4}$$

The goodness of fit for the regression equation is normally measured using the coefficient of determination, $R^2$, which is given by:

$$R^2 = \frac{\sum (Y_i - \bar{y})^2}{\sum (y_i - \bar{y})^2} \tag{5.5}$$

Where $\bar{y}$ = Average position of the original data;

$y_i$ = Original position of point $i$; and

$Y_i$ = Estimated position of point $i$ using Equation 5.2.

The coefficient of determination ranges from 0 to 1; a value closer to 1 indicates a greater degree of correlation.

### 5.2.1c Curve Fitting

For the curved segment of the alignment, a circular curve was first fitted and then the remaining points at both ends of the circular curve were treated as transition curves. The equation for fitting circular curve is

$$\sqrt{(x_i - x_0)^2 + (y_i - y_0)^2} - r = 0 \tag{5.6}$$

Where $(x_0, y_0)$ = Coordinates of the center of circular curve; and

$r$ = Radius of the circle.

These three unknowns can be determined using non-linear regression through multiple iterations (Manthey 1999). An initial guess for the iteration process is based only on the middle half of the points on the curve so that this initial estimate would not be disturbed by any points that may have been part of the transition curves before and after the circular curve. Using the first, middle and last point of the selected list of points (middle half) the equation of the circumscribed circle is applied to find $x_0$, $y_0$ and $r$. As shown in Figure 5.1, if the three points have coordinates $(x_1, y_1), (x_2, y_2)$ and $(x_3, y_3)$, the equations can be written as (Rowland, 2000):

$$x_0 = \frac{d(a^2 + c^2) - c(b^2 + d^2)}{2(ad - bc)} \tag{5.7}$$

$$y_0 = \frac{a(b^2 + d^2) - b(a^2 + c^2)}{2(ad - bc)} \tag{5.8}$$

$$r = \sqrt{(x_1 - x_0)^2 + (y_1 - y_0)^2} \tag{5.9}$$

Where
$$a = x_2 - x_1$$
$$b = x_3 - x_1$$
$$c = y_2 - y_1$$
$$d = y_3 - y_1$$

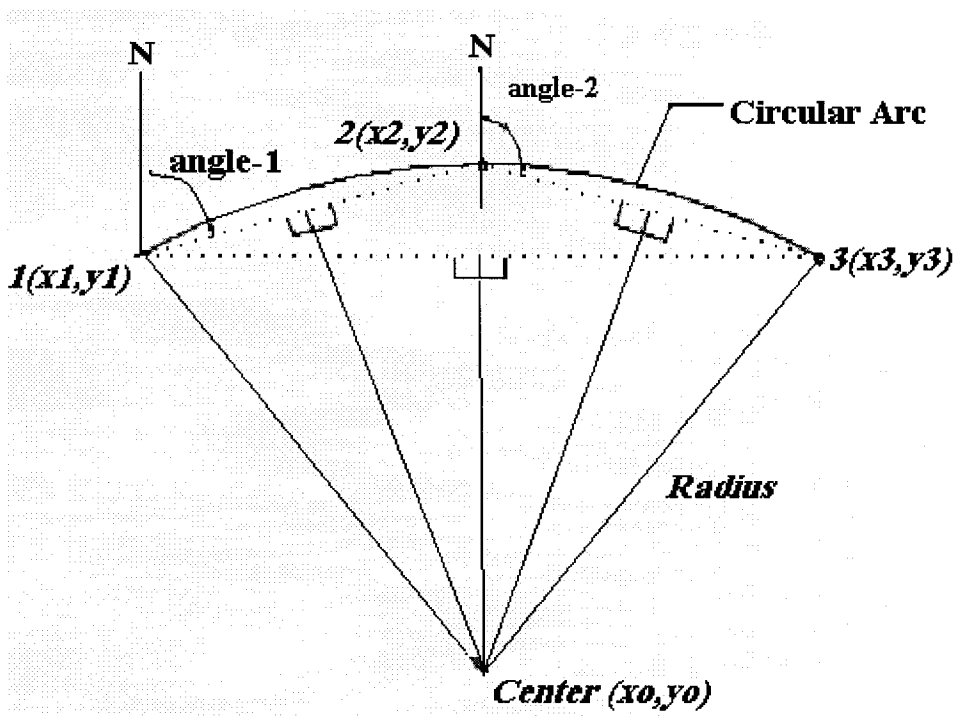



**Figure 5.1: First, Middle and Last Point in Middle 50% Data Points of Curved Segment.**

The values of ($x_0$, $y_0$) and $r$ determined using Equations 5.7, 5.8 and 5.9 are used as a first guess in the Jacobian matrix method for solving the non-linear equation of a circle (Equation 5.6), where the Jacobian matrix, $J$, is formulated as:

$$J = \begin{bmatrix} \frac{\partial F_1}{\partial x_0} & \frac{\partial F_1}{\partial y_0} & \frac{\partial F_1}{\partial r} \\ \frac{\partial F_2}{\partial x_0} & \frac{\partial F_2}{\partial y_0} & \frac{\partial F_2}{\partial r} \\ \frac{\partial F_3}{\partial x_0} & \frac{\partial F_3}{\partial y_0} & \frac{\partial F_3}{\partial r} \\ \vdots & \vdots & \vdots \end{bmatrix} \tag{5.10}$$

$$F = \sqrt{(x_i - x_0)^2 + (y_i - y_0)^2} - r \tag{5.11}$$

$$\frac{\partial F_i}{\partial x_0} = \frac{x_0 - x_i}{\sqrt{x_i^2 - 2x_0x_i + y_i^2 - 2y_0y_i + x_0^2 + y_0^2}} \tag{5.12}$$

$$\frac{\partial F_i}{\partial y_0} = \frac{y_0 - y_i}{\sqrt{x_i^2 - 2x_0x_i + y_i^2 - 2y_0y_i + x_0^2 + y_0^2}} \tag{5.13}$$

$$\frac{\partial F_i}{\partial r} = -1 \tag{5.14}$$

A residual matrix, $K$, is then calculated as:

$$K = \begin{bmatrix} 0 - \left(\sqrt{(x_1 - x_0)^2 + (y_1 - y_0)^2} - r\right) \\ 0 - \left(\sqrt{(x_2 - x_0)^2 + (y_2 - y_0)^2} - r\right) \\ 0 - \left(\sqrt{(x_3 - x_0)^2 + (y_3 - y_0)^2} - r\right) \\ \vdots \end{bmatrix} \tag{5.15}$$

Adjustments for the three unknowns, $(x_0, y_0)$ and $r$ are then calculated as $\Delta x_0$, $\Delta y_0$, and $\Delta r$, using the following equation:

$$\begin{bmatrix} \Delta x_0 \\ \Delta y_0 \\ \Delta r \end{bmatrix} = (J^T J)^{-1} J^T K \tag{5.16}$$

The adjustments, $\Delta x_0$, $\Delta y_0$, and $\Delta r$, are added to the first guess, and the resulting values are then used in a second iteration. Further iterations are performed until the adjustment values become close to zero.

### 5.2.1d Algorithm

The algorithm used for development of Roadfit can be summarized as follows:

1. Consider a curve with straight entry and exit segments at both ends. The curve includes a circular arc and may include a spiral curve.
2. Determine a first estimate of the points falling along the straight start/end segments within a certain angular tolerance.
3. The remaining points belong to the curved segment and may include spiral arcs.
4. Take the first 75% of the points lying along the straight segments (to ensure that spirals are not included). Perform linear regression on these points to get a line of best fit. Extend this line to the last point of the straight points list.
5. Add the first point of the curve list as the last point of the entry tangent and add first point of the exit tangent as the last point of the curved segment.
6. Determine the angles made by both entry and exit tangents.
7. Draw the entry and exit tangents.

8. Take the middle 50% points of the curved segment. Use the first, middle and last point from this list (middle 50%) and find an initial estimate of the circle.
9. Using this initial estimate as a first guess; perform circular non-linear regression on the curve points.
10. With the values obtained from 9, draw a circle.
11. Add all those points to the circle that lie either at any distance inside the circle or outside the circle within a specific offset distance *d* (Figure 5.2).
12. Add all those points to the tangents that lie on either side within a specific offset distance *d* (Figure 5.2).
13. The points beyond the offset distance *d* (from both tangent and circle) are added to a separate anomalous list.
14. From the anomalous list, take the points lying between the end of entry tangent and start of circle, and consider them as entry spiral. Similarly points between the end of circle and start of exit tangent make the exit spiral.
15. Find length, radius, and center of the circular curve.
16. Find length of the spiral.

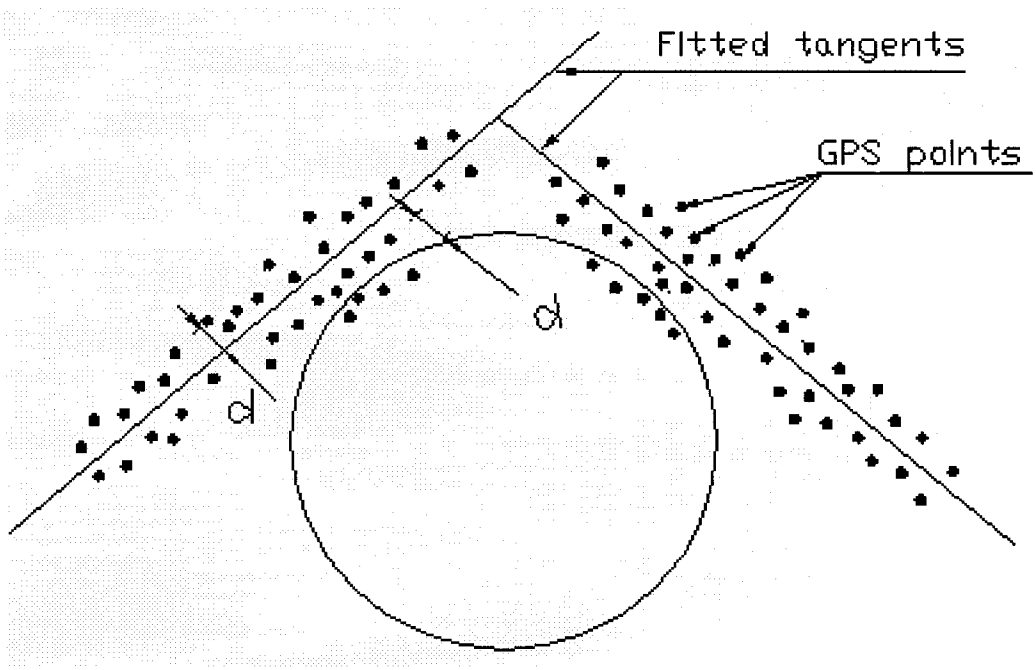



**Figure 5.2: Locating Transition Curve between Tangent and Circular Arc, Beyond the Offset Distance *d* from Tangent and Circle.**

The output from Roadfit includes the radius of circular curve, length of circular curve, coordinates of the circular curve center, length of entry and exit spiral, average spacing between points, and coefficient of determination, $R^2$ (Figure 5.3). The developed Extension was verified using the real GPS data collected for horizontal alignment of Highway 31 and Highway 41.

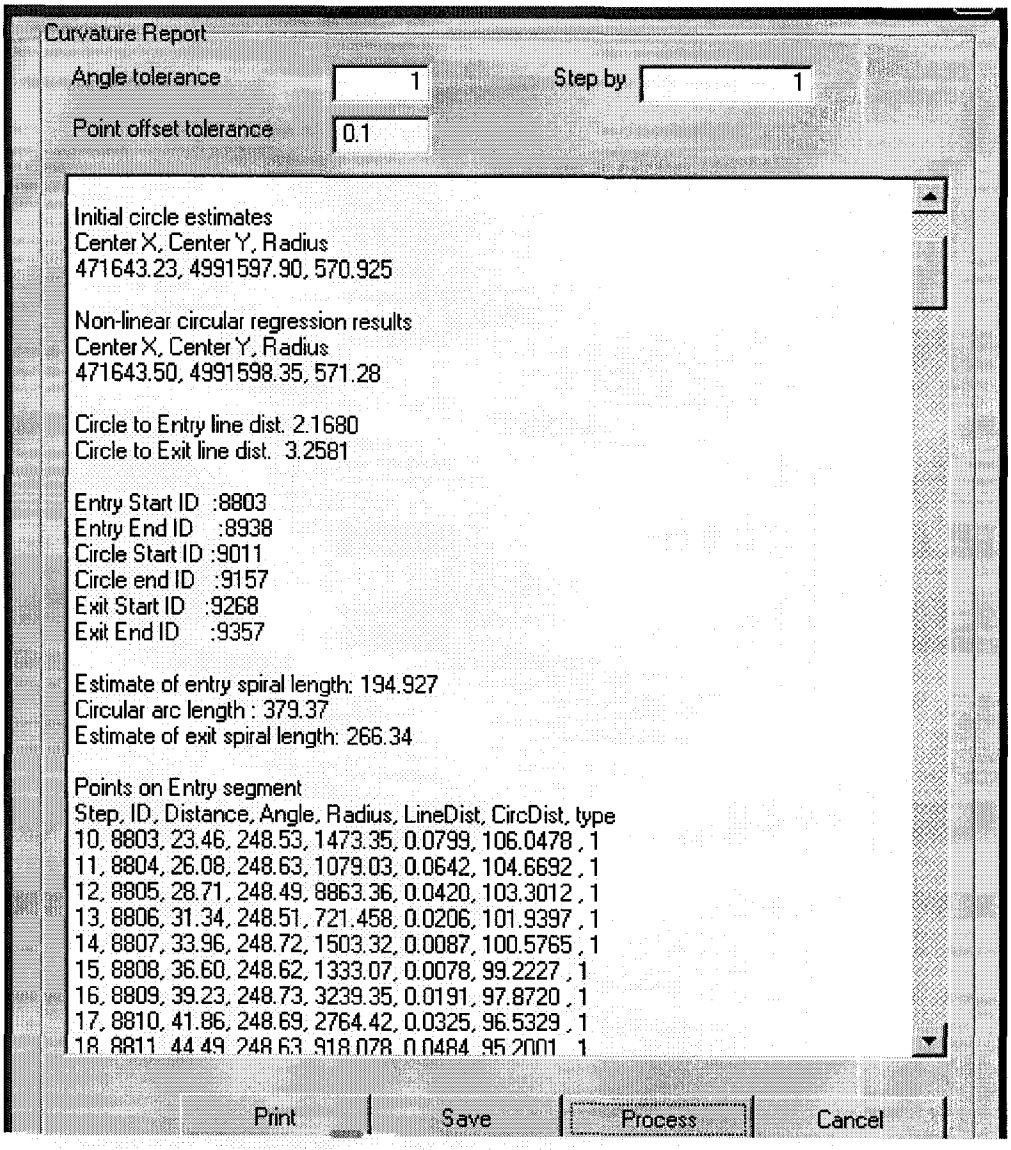



**Figure 5.3: Output Interface of the Developed Extension.**

## 5.3 Sensitivity Analysis

### 5.3.1 Fitting Parameters

Sensitivity analysis was carried out to examine the extent to which the output of Roadfit depends upon the input parameters. This is an important step for checking the quality of a model, as well as a powerful tool for checking the robustness and reliability

of its analysis. As discussed in the previous section, three parameters are used in Roadfit to categorize the different features of a highway alignment. The first parameter to distinguish between tangent and curve is the angle tolerance ($\delta$). The second is the offset distance ($d$) from the fitted circular arc and straight segment to the remaining scattered points. The points beyond this distance are considered to belong to a transition curve. The third parameter is the number of points to skip (*nps*) during calculating the angle $\theta_i$. It was noted that calculating $\theta_i$ using successive points with a very short spacing might produce inconsistent trends. Therefore, Roadfit was developed with the capability of skipping intermediate points in the angle calculation. A sensitivity analysis was carried out to suggest optimum values of these three parameters in the application of Roadfit.

As shown in Table 5.1, twelve trials were carried out. In the first set of four trials, the angle tolerance and offset distance are kept constant to a comparatively low value of $0.5^{\circ}$ and 0.05 m, respectively. In the second set of four trials, the parameters $\delta$ and $d$ are increased to $1^{\circ}$ and 0.1 m, respectively. Similarly, in the final set of four trials, $\delta$ and $d$ are increased to $2^{\circ}$ and 0.20 m, respectively. The third parameter, *nps*, was set to 0, 1, 2, and 5 for all four trials for each set. All nine curves on Highway 31 were analyzed with each trial using the real GPS data collected at 0.10 second observation interval while traveling at 80 km/h. Both southward and northward alignments were analyzed separately.

The geometric features, including radius and length of circular curve as well as length of entry and exit spirals ($L_{s1}$ and $L_{s2}$) obtained from the fitted alignment were compared to the corresponding features of the actual highway alignment. The absolute difference between the fitted and actual values was calculated for each curve along with

the percentage change in features of the fitted alignment with respect to actual alignment defined as:

$$\text{Percentage Difference} = \frac{|\text{Observed value} - \text{Actual value}|}{\text{Actual value}} * 100 \tag{5.17}$$

After compiling the results for all twelve trials, the average percent differences obtained for the southward alignment are compared with those of the northward alignment (Table 5.2). Similarly, the standard deviation values are also compared for both directions of the alignment as a measure of the variability of the absolute difference between actual and fitted alignment (Table 5.3).

**Table 5.1: Selection of parameters for various trials**

| | **Trial Number** | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Variable** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **12** |
| **$\delta^o$** | 0.50 | 0.50 | 0.50 | 0.50 | 1.00 | 1.00 | 1.00 | 1.00 | 2 | 2 | 2 | 2 |
| ***d* (m)** | 0.05 | 0.05 | 0.05 | 0.05 | 0.10 | 0.10 | 0.10 | 0.10 | 0.20 | 0.20 | 0.20 | 0.20 |
| ***nps*** | 0 | 1 | 2 | 5 | 0 | 1 | 2 | 5 | 0 | 1 | 2 | 5 |

**Table 5.2: Average Percent Difference between Actual and Observed Alignment**

| Trial | Southward Alignment | | | | Northward Alignment | | | | Average Two Directions | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ |
| 1 | 8.11 | 46.26 | 48.59 | 68.68 | 2.75 | 33.74 | 45.58 | 79.03 | 5.43 | 40.00 | 47.09 | 73.86 |
| 2 | 8.11 | 46.26 | 48.59 | 68.68 | 2.75 | 33.74 | 45.58 | 78.00 | 5.43 | 40.00 | 47.09 | 73.34 |
| 3 | 8.11 | 46.26 | 48.59 | 67.45 | 2.75 | 33.74 | 45.58 | 78.53 | 5.43 | 40.00 | 47.09 | 72.99 |
| 4 | 8.11 | 46.26 | 48.59 | 67.53 | 2.72 | 33.58 | 45.58 | 79.00 | 5.42 | 39.92 | 47.09 | 73.27 |
| 5 | 3.10 | 27.72 | 49.94 | 100.10 | 2.62 | 26.73 | 73.50 | 80.50 | 2.86 | 27.23 | 61.72 | 90.30 |
| 6 | 3.10 | 27.72 | 49.94 | 101.19 | 2.62 | 26.73 | 73.50 | 79.87 | 2.86 | 27.23 | 61.72 | 90.53 |
| 7 | 3.10 | 27.72 | 49.94 | 102.08 | 2.61 | 26.73 | 73.50 | 82.20 | 2.86 | 27.23 | 61.72 | 92.14 |
| 8 | 3.10 | 27.72 | 49.94 | 102.73 | 2.58 | 26.51 | 72.68 | 82.64 | 2.84 | 27.12 | 61.31 | 92.69 |
| 9 | 3.30 | 17.26 | 69.61 | 85.68 | 3.88 | 34.85 | 98.83 | 99.99 | 3.59 | 26.06 | 84.22 | 92.84 |
| 10 | 3.30 | 17.26 | 69.61 | 86.45 | 3.88 | 34.85 | 98.83 | 100.00 | 3.59 | 26.06 | 84.22 | 93.23 |
| 11 | 3.30 | 17.26 | 69.61 | 86.46 | 3.88 | 34.85 | 98.83 | 101.75 | 3.59 | 26.06 | 84.22 | 94.11 |
| 12 | 3.30 | 17.26 | 69.61 | 86.45 | 3.88 | 34.85 | 98.83 | 104.55 | 3.59 | 26.06 | 84.22 | 95.50 |
| **Minimum** | **3.10** | **17.26** | **48.59** | **67.45** | **2.58** | **26.51** | **45.58** | **78.00** | **2.84** | **26.06** | **47.09** | **72.99** |

**$r$ = Radius of Curve** **$L_c$ = Length of Curve** **$L_{s1}$ = Length of Entry Spiral** **$L_{s2}$ = Length of Exit Spiral**

**Table 5.3: Average Standard Deviation for Percent Difference between Actual and Observed Alignment**

| Trial | Southward Alignment | | | | Northward Alignment | | | | Average Two Directions | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ |
| 1 | 11.92 | 52.87 | 55.43 | 51.38 | 3.19 | 23.93 | 33.80 | 63.15 | 7.56 | 38.40 | 44.62 | 57.27 |
| 2 | 11.92 | 52.87 | 55.43 | 51.38 | 3.19 | 23.93 | 33.80 | 63.31 | 7.56 | 38.40 | 44.62 | 57.35 |
| 3 | 11.93 | 52.87 | 55.43 | 52.33 | 3.19 | 23.93 | 33.80 | 62.47 | 7.56 | 38.40 | 44.62 | 57.40 |
| 4 | 11.93 | 52.87 | 55.43 | 54.16 | 3.20 | 24.04 | 32.77 | 61.25 | 7.57 | 38.46 | 44.10 | 57.71 |
| 5 | 3.23 | 27.90 | 39.97 | 96.77 | 2.40 | 16.54 | 19.50 | 64.68 | 2.82 | 22.22 | 29.74 | 80.73 |
| 6 | 3.23 | 27.90 | 39.97 | 99.22 | 2.40 | 16.54 | 19.50 | 64.44 | 2.82 | 22.22 | 29.74 | 81.83 |
| 7 | 3.23 | 27.90 | 39.97 | 110.82 | 2.41 | 16.54 | 19.50 | 58.44 | 2.82 | 22.22 | 29.74 | 84.63 |
| 8 | 3.23 | 27.90 | 39.97 | 111.72 | 2.42 | 16.63 | 19.28 | 60.77 | 2.83 | 22.27 | 29.63 | 86.25 |
| 9 | 2.88 | 14.09 | 41.85 | 92.04 | 3.47 | 18.05 | 99.47 | 83.11 | 3.18 | 16.07 | 70.66 | 87.58 |
| 10 | 2.88 | 14.09 | 41.85 | 91.22 | 3.47 | 18.05 | 99.47 | 81.90 | 3.18 | 16.07 | 70.66 | 86.56 |
| 11 | 2.88 | 14.09 | 41.85 | 93.27 | 3.47 | 18.05 | 99.47 | 87.32 | 3.18 | 16.07 | 70.66 | 90.30 |
| 12 | 2.88 | 14.09 | 41.85 | 95.24 | 3.47 | 18.05 | 99.47 | 85.37 | 3.18 | 16.07 | 70.66 | 90.31 |
| **Minimum** | **2.88** | **14.09** | **39.97** | **51.38** | **2.40** | **16.54** | **19.28** | **58.44** | **2.82** | **16.07** | **29.63** | **57.27** |

From the sensitivity analysis it is observed that changing *nps* has no significant effect on the output. However, the output of Roadfit is more sensitive to variations in the values of the angle tolerance $\delta$ and offset distance $d$. At the lower values of $\delta$=0.50° and $d$=0.05 m, the results of the fitted alignment are quite poor as reflected in the higher values of average differences between the actual and fitted alignment. The results were improved (difference between actual and fitted alignment decreased) when the values of $\delta$ and $d$ were increased to 1° and 0.1 m respectively. Further increase in $\delta$ and $d$ to the values of 2° and 0.20 m, respectively, caused an increase in the difference between actual and fitted alignments. Although the standard deviations for higher values of $\delta$ and $d$ were better, the final selection of the parameters was made on the basis of absolute average difference between the actual and fitted alignment. Based on this sensitivity analysis, the parameters were optimized to $\delta$=1°, *nps*=1, and $d$=0.1 m.

### 5.3.2 Observation Interval

After fixing the three parameters for Roadfit, a comparison was also made between the results based upon 0.1, 0.5 and 1.0 second as the observation interval for GPS data sets. As mentioned earlier, the original data were collected at a 0.1-second interval or a rate of 10 points per second. In ArcView, a small script was developed to change the observation interval. Using this script, two additional sets of coordinate data were created with GPS points at 0.5 and 1 second interval. These data sets were analyzed for finding the required parameters of all nine curves in both directions of the alignment of Highway 31 as observed at 80 km/h speed.

To check the sensitivity of the observation interval with the number of points to skip (*nps*), the latter was again taken as 0, 1, 2, and 5 for each run of the analysis. Eight more sets of results, four for each new observation interval, were compiled and compared to each other and to another set of four trials corresponding to the original interval of 0.1-second (Table 5.4). The average of the percentage difference between actual and fitted alignment for all the nine curves are summarised in Table 5.5, while the standard deviations are given in Table 5.6. It was found that increasing the observation interval reduces the quality of results. This is clearly indicated by the larger difference between actual and observed values of the data set with 1.0-second interval. In other words coordinates recorded with smaller spacing give better results than those with larger spacing. Figure 5.4 shows a graphic comparison between actual and fitted radii for all the three types of observation interval.

The higher observation intervals (0.5 and 1 second) were also sensitive to the number of points to skip. During processing, when skipping 5 points, the script could not find sufficient points to estimate the length of spiral at either one end or in some cases at both ends of the circular curve. In such circumstances the length of the selected segment of the highway must be increased. Further analysis of the results is based on the data observed at 0.1 second interval.

**Table 5.4: Selection of Parameters for Various Trials**

| Variable | Trial Number | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **5** | **6** | **7** | **8** | **13** | **14** | **15** | **16** | **17** | **18** | **19** | **20** |
| **Interval (second)** | 0.1 | 0.1 | 0.1 | 0.1 | 0.5 | 0.5 | 0.5 | 0.5 | 1 | 1 | 1 | 1 |
| **δ°** | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| ***d* (m)** | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 |
| ***nps*** | 0 | 1 | 2 | 5 | 0 | 1 | 2 | 5 | 0 | 1 | 2 | 5 |

**Table 5.5: Average Percent Difference between Actual and Observed Alignment**

| Trial | Southward Alignment | | | | Northward Alignment | | | | Average Two Directions | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ |
| **1** | 3.10 | 27.72 | 49.94 | 100.10 | 2.62 | 26.73 | 73.50 | 80.50 | 2.86 | 27.23 | 61.72 | 90.30 |
| **2** | 3.10 | 27.72 | 49.94 | 101.19 | 2.62 | 26.73 | 73.50 | 79.87 | 2.86 | 27.23 | 61.72 | 90.53 |
| **3** | 3.10 | 27.72 | 49.94 | 102.08 | 2.61 | 26.73 | 73.50 | 82.20 | 2.86 | 27.23 | 61.72 | 92.14 |
| **4** | 3.10 | 27.72 | 49.94 | 102.73 | 2.58 | 26.51 | 72.68 | 82.64 | 2.84 | 27.12 | 61.31 | 92.69 |
| **5** | 2.79 | 17.96 | 72.80 | 65.57 | 8.38 | 40.84 | 98.90 | 91.03 | 5.59 | 29.40 | 85.85 | 78.30 |
| **6** | 2.79 | 17.96 | 72.80 | 65.57 | 8.38 | 40.84 | 98.90 | 88.73 | 5.59 | 29.40 | 85.85 | 77.15 |
| **7** | 2.79 | 17.96 | 72.80 | 57.45 | 8.38 | 40.84 | 98.90 | 102.64 | 5.59 | 29.40 | 85.85 | 80.05 |
| **8** | 2.79 | 17.96 | 72.80 | 82.97 | 5.30 | 34.42 | 88.71 | 93.41 | 4.05 | 26.19 | 80.76 | 88.19 |
| **9** | 2.68 | 22.78 | 195.83 | 157.83 | 4.26 | 27.89 | 175.49 | 126.51 | 3.47 | 25.34 | 185.66 | 142.17 |
| **10** | 2.68 | 22.78 | 195.83 | 157.19 | 6.40 | 36.74 | 175.49 | 97.38 | 4.54 | 29.76 | 185.66 | 127.29 |
| **11** | 2.68 | 22.78 | 195.83 | 162.65 | 4.26 | 27.89 | 175.49 | 132.01 | 3.47 | 25.34 | 185.66 | 147.33 |
| **12** | 3.08 | 22.66 | 181.56 | 203.92 | 4.32 | 28.47 | 241.40 | 114.02 | 3.70 | 25.57 | 211.48 | 158.97 |
| **Minimum** | **2.68** | **17.96** | **49.94** | **57.45** | **2.58** | **26.51** | **72.68** | **79.87** | **2.84** | **25.34** | **61.31** | **77.15** |

**Table 5.6: Average Standard Deviation for Percent Difference between Actual and Observed Alignment**

| Trial | Southward Alignment | | | | Northward Alignment | | | | Average Two Directions | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | $r$ | $L_c$ | $L_{s1}$ | $L_{s2}$ |
| **1** | 3.23 | 27.90 | 39.97 | 96.77 | 2.40 | 16.54 | 19.50 | 64.68 | 2.82 | 22.22 | 29.74 | 80.73 |
| **2** | 3.23 | 27.90 | 39.97 | 99.22 | 2.40 | 16.54 | 19.50 | 64.44 | 2.82 | 22.22 | 29.74 | 81.83 |
| **3** | 3.23 | 27.90 | 39.97 | 110.82 | 2.41 | 16.54 | 19.50 | 58.44 | 2.82 | 22.22 | 29.74 | 84.63 |
| **4** | 3.23 | 27.90 | 39.97 | 111.72 | 2.42 | 16.63 | 19.28 | 60.77 | 2.83 | 22.27 | 29.63 | 86.25 |
| **5** | 2.28 | 9.47 | 60.26 | 58.33 | 13.24 | 47.87 | 86.86 | 89.84 | 7.76 | 28.67 | 73.56 | 74.09 |
| **6** | 2.28 | 9.47 | 60.26 | 58.33 | 13.24 | 47.87 | 86.86 | 86.25 | 7.76 | 28.67 | 73.56 | 72.29 |
| **7** | 2.28 | 9.47 | 60.26 | 55.46 | 13.24 | 47.87 | 86.86 | 100.15 | 7.76 | 28.67 | 73.56 | 77.81 |
| **8** | 2.28 | 9.47 | 60.26 | 75.86 | 5.26 | 30.63 | 78.01 | 83.45 | 3.77 | 20.05 | 69.14 | 79.66 |
| **9** | 1.89 | 18.50 | 118.76 | 107.14 | 3.96 | 16.26 | 164.55 | 129.58 | 2.93 | 17.38 | 141.66 | 118.36 |
| **10** | 1.89 | 18.50 | 118.76 | 128.41 | 7.43 | 30.60 | 164.55 | 122.04 | 4.66 | 24.55 | 141.66 | 125.23 |
| **11** | 1.89 | 18.50 | 118.76 | 138.42 | 3.96 | 16.26 | 164.55 | 128.24 | 2.93 | 17.38 | 141.66 | 133.33 |
| **12** | 2.99 | 20.09 | 123.35 | 101.57 | 4.02 | 17.70 | 202.63 | 110.89 | 3.51 | 18.90 | 162.99 | 106.23 |
| **Minimum** | **1.89** | **9.47** | **39.97** | **55.46** | **2.40** | **16.26** | **19.28** | **58.44** | **2.82** | **17.38** | **29.63** | **72.29** |

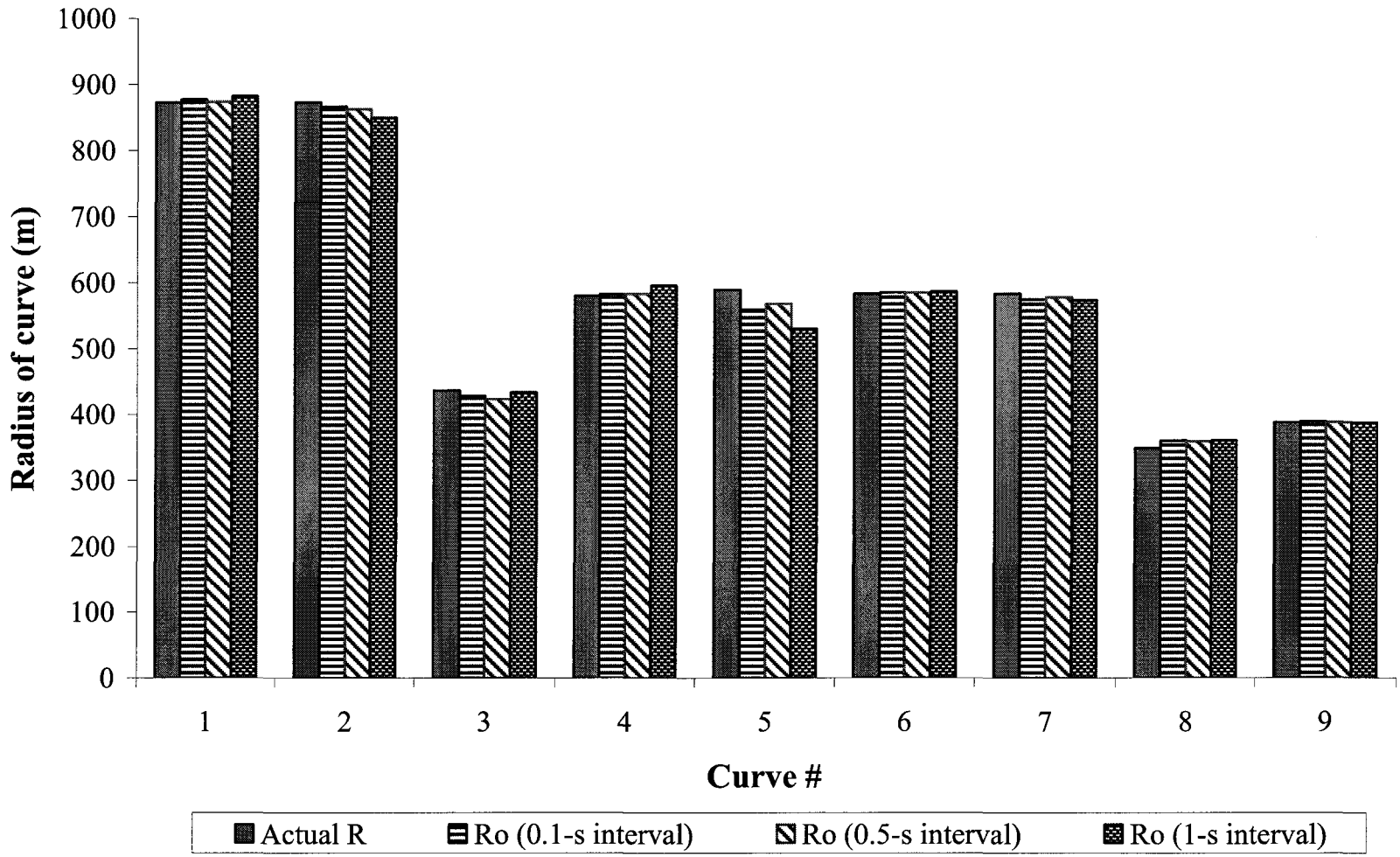



**Figure 5.4: Comparison of Actual and Fitted Radii at Different Observation Interval.**

## 5.4 Analysis of Fitted Alignment

Based on the results of the sensitivity analysis, the fitting parameters were set as $\delta=1^{\circ}$, $nps=1$, and $d=0.1$ m. Roadfit was then employed to fit the horizontal alignment of both Highways 31 and 41 using the collected GPS data at 0.1 second observation interval. The post-processed GPS data were loaded to ArcView in delimited text format as a new event theme in the project window. The data were divided into twelve files; each file representing an individual run of the selected highway segment corresponding to a specific speed (80, 90, or 100 km/h) and direction of travel. Each of the twelve themes/files was then converted to shapefile so that it could be processed for further analysis. Roadfit was applied to each of the twelve files individually, and the geometric features of each curve were determined for all twelve fitted alignments (six for Highway 31 and six for Highway 41).

Starting from one end of the selected highway segment and using "select feature" button "", each curve was selected independently along with some part of the tangents at both ends of the curve. Thus, the selection included two tangents, two spirals and a circular curve. The selected segment was analyzed to determine the required geometric features of the horizontal alignment. Two types of results were determined:

1. The geometric features of observed horizontal alignments including radii and lengths of circular curves and lengths of transition curves at both ends of the circular arcs,
2. The lateral distance between the actual centerline of highway and the observed trajectory of vehicle.

During the analysis, two curves had extra points running parallel to the trajectory being determined. These points seemed to occur when it was not possible to maintain the desired speed due to either a slow moving vehicle ahead or some other hindrance. In such cases, the driver usually pulled over the car and returned to the original position and started again resulting in a duplicate track up to that location. These extra unwanted points were deleted in the corresponding attribute table.

### 5.4.1 Comparison of Curve Radius

Table 5.7 contains the observed radius for each curve at all the three speeds and with each driving direction for Highway 31. These results are quite encouraging and are very close to the actual values. There is only 1.34 % average difference between the observed and actual values, and the average standard deviation in the data is 1.58. A reason for this difference may be attributed to the discrepancy between the drawing's values and the site values. Such errors are usually caused during measurements for developing the as-built drawings of the highway. Also the as-built drawings were developed in the years 1975 and 1976, and chances of on-site changes with time should not be ignored. In other words it may be assumed that observed values are equal to the actual site values but slightly different than design values. The vehicle's trajectory on Curve 5 is significantly different from other curves. The fitted radius for Curve 5 is 4.83% smaller than the actual radius. This significant difference could be due to the existence of Curve 6 in close vicinity. Similarly, the driver traversed the sharpest curve (Curve 8) in the sample with a radius 3.11% larger than actual radius. This difference in the observed and actual radius indicates inconsistency in the actual curve radius. No

significant effect of the speed or direction of driving was observed. As shown in Table 5.8, the average difference between actual and observed radius for Highway 41 is higher (3%) than that for Highway 31. It may be due to the relatively congested environment and mountainous topography of Highway 41.

### 5.4.2 Comparison of Curve Length

Tables 5.9 and 5.10 compare the lengths of curves of fitted alignments with that of actual alignment. The shortest curves of Highway 31 (Curves 3 and 4) have been tracked with longer lengths (24.55 and 36.51% longer than actual) compared to other curves of the same highway. Similarly, in Highway 41, the fitted lengths for the shortest curves (Curves 2, 6 and 7) are significantly higher than the actual values. On other hand the longest curve of Highway 31 (Curve 7) was traversed with a length shorter than actual value by 15.21%. Tables 5.11 and 5.12 compare sum of the lengths of fitted circular and spiral curves (average for all three speeds and both travel directions) with the corresponding sum of actual circular and spiral curves for Highways 31 and 41, respectively. These tables also compare average lengths of entry and exit spirals for the two types of alignments. The average difference in total length for Highway 31 is only 12.39%, which is quite better than the difference in individual lengths. The smaller difference in total length also shows that the driver followed some part of the circular curves with a spiral path (particularly Curve 7).

Although the driver was following the middle of the lane, the trends presented by the observed alignment indicate that he could not exactly track the centerline of the lane along the curves. This is not only a logical behaviour of the driver but also agrees with

the claims made by various researchers about the discrepancies involved in the geometric design of highways. A reason for the relatively larger difference in length could be the difference in drawing's values and the corresponding site values. It is also worth mentioning that from a safety perspective, the length of curve is less significant as compared to radius of the curve. The effect of speed on curve length is also summarised in Figure 5.5. Results obtained at 80 km/h speed are comparatively closer to the actual values. Hence for tracking highway alignment more accurately, the lower speed of 80 km/h was better than speeds of 90 and 100 km/h.

### 5.4.3 Comparison of Lateral Displacement

To determine the lateral displacement between actual alignment and the observed alignment, digitized maps of Highways 31 and 41 built by DMTI (Desktop Mapping Technologies Inc.) were used. The original DMTI maps were in line shapes which were converted to point shape with a point spacing of 10 meters. Moreover, the DMTI maps were built in NAD-83 datum (North American Datum), whereas the observed alignments were in WGS-84 (World Geodetic System). The DMTI maps were translated to WGS-84 using the FME software, so that both types of data had the same datum. According to the DMTI sources, the maps were compiled based on aerial survey data with an error of ± 10 meters.

**Table 5.7: Comparison of Actual Radius with Observed Radius for Highway 31**

| Curve | Radius (m) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Actual | Observed (Ro) | | | | | | | Difference | |
| | R | S80 | S90 | S100 | N80 | N90 | N100 | Average | Meters | %(absolut) |
| 1 | 873.20 | 875.47 | 886.09 | 855.46 | 889.08 | 892.33 | 884.39 | 880.47 | 7.27 | 0.83 |
| 2 | 873.20 | 844.00 | 866.84 | 869.50 | 904.27 | 843.42 | 893.44 | 870.25 | -2.96 | 0.34 |
| 3 | 436.60 | 478.09 | 422.29 | 416.13 | 446.29 | 417.53 | 421.17 | 433.58 | -3.02 | 0.69 |
| 4 | 580.00 | 553.52 | 585.81 | 576.02 | 530.71 | 625.74 | 607.19 | 579.83 | -0.17 | 0.03 |
| 5 | 589.00 | 586.22 | 545.87 | 544.40 | 570.53 | 568.94 | 547.26 | 560.54 | -28.46 | 4.83 |
| 6 | 583.00 | 573.74 | 592.62 | 583.25 | 590.63 | 582.45 | 590.46 | 585.53 | 2.53 | 0.43 |
| 7 | 583.00 | 583.92 | 578.79 | 577.40 | 574.87 | 571.28 | 578.27 | 577.42 | -5.58 | 0.96 |
| 8 | 349.00 | 371.86 | 374.90 | 356.50 | 351.47 | 355.88 | 348.58 | 359.87 | 10.87 | 3.11 |
| 9 | 388.00 | 393.67 | 392.69 | 391.39 | 391.57 | 396.87 | 380.78 | 391.16 | 3.16 | 0.81 |
| **Average Difference** | | | | | | | | | **-1.82** | **1.34** |
| **Standard Deviation** | | | | | | | | | **11.27** | **1.58** |

**Note:** N, S indicates driving direction to be North or South respectively.

80, 90 and 100 indicates speed of car in km/h.

**Table 5.8: Comparison of Actual Radius with Observed Radius for Highway 41**

| | Radius (m) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Actual | Observed (Ro) | | | | | | | Difference | |
| Curve | R | N80 | N90 | N100 | S80 | S90 | S100 | Average | Meters | %(absolute) |
| 1 | 388.00 | 378.16 | 369.65 | 396.95 | LoS | 418.74 | 397.31 | 392.16 | 4.16 | 1.07 |
| 2 | 582.12 | LoS | 591.37 | 528.06 | LoS | 594.55 | 557.15 | 567.78 | -14.34 | 2.46 |
| 3 | 873.19 | 726.79 | 852.01 | 853.69 | 862.50 | 886.30 | 855.35 | 839.44 | -33.75 | 3.87 |
| 4 | 873.19 | 937.22 | 858.81 | 917.22 | 872.68 | 890.64 | 978.91 | 909.25 | 36.06 | 4.13 |
| 5 | 806.00 | 825.45 | 822.46 | 809.47 | 775.97 | 754.65 | 564.57 | 758.76 | -47.24 | 5.86 |
| 6 | 582.12 | 634.35 | 559.39 | 600.51 | 636.87 | 549.74 | 557.46 | 589.72 | 7.60 | 1.31 |
| 7 | 436.59 | 452.84 | 486.39 | 408.85 | 458.35 | LoS | 450.70 | 451.43 | 14.84 | 3.40 |
| 8 | 436.59 | 381.91 | 456.55 | 436.41 | 441.50 | 405.44 | 448.44 | 428.38 | -8.21 | 1.88 |
| **Average Difference** | | | | | | | | | **-5.11** | **3.00** |
| **Standard Deviation** | | | | | | | | | **26.81** | **1.63** |

LoS = Loss of Signal

**Table 5.9: Comparison of Actual Length of Curve with Observed Length for Highway 31**

| Curve | Length (m) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Actual | Observed (Lo) | | | | | | | Difference | |
| | Lc | S80 | S90 | S100 | N80 | N90 | N100 | Average | Meters | %(absolute) |
| 1 | 464.20 | 531.62 | 580.57 | 286.13 | 562.52 | 585.04 | 551.39 | 516.21 | 52.01 | 11.20 |
| 2 | 474.00 | 263.66 | 504.41 | 463.25 | 641.18 | 269.73 | 569.17 | 451.90 | -22.10 | 4.66 |
| 3 | 162.40 | 306.23 | 154.34 | 141.73 | 261.63 | 161.23 | 188.44 | 202.27 | 39.87 | 24.55 |
| 4 | 170.00 | 184.29 | 243.36 | 238.19 | 196.94 | 287.09 | 242.57 | 232.07 | 62.07 | 36.51 |
| 5 | 341.25 | 468.21 | 268.05 | 321.56 | 224.58 | 285.72 | 238.28 | 301.07 | -40.18 | 11.77 |
| 6 | 377.33 | 363.17 | 423.67 | 377.34 | 445.69 | 444.41 | 449.56 | 417.31 | 39.98 | 10.59 |
| 7 | 783.64 | 787.11 | 481.63 | 605.45 | 524.72 | 379.37 | 554.65 | 555.49 | -228.15 | 29.11 |
| 8 | 472.38 | 652.29 | 663.07 | 464.12 | 456.60 | 585.23 | 301.55 | 520.48 | 48.10 | 10.18 |
| 9 | 527.00 | 452.70 | 616.72 | 603.21 | 624.54 | 682.41 | 290.9 | 545.08 | 18.08 | 3.43 |
| **Average Difference** | | | | | | | | | **-3.37** | **15.78** |
| **Standard Deviation** | | | | | | | | | **66.69** | **16.70** |

**Table 5.10: Comparison of Actual Length of Curve with Observed Length for Highway 41**

| Curve | Length (m) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Actual | Observed (Lo) | | | | | | | Difference | |
| | Lc | N80 | N90 | N100 | S80 | S90 | S100 | Average | Meters | %(absolute) |
| 1 | 159.79 | 272.41 | 161.77 | 227.21 | LoS | 259.13 | 173.89 | 218.88 | 59.09 | 36.98 |
| 2 | 110.91 | LoS | 326.03 | 127.69 | LoS | 176.46 | 162.68 | 198.22 | 87.31 | 78.72 |
| 3 | 430.66 | 94.07 | 477.02 | 433.61 | 414.13 | 498.92 | 273.09 | 365.14 | -65.52 | 15.21 |
| 4 | 208.79 | 167.93 | 281.32 | 276.48 | 224.55 | 282.10 | 126.79 | 226.53 | 17.74 | 8.50 |
| 5 | 541.13 | 663.85 | 597.38 | 707.89 | 529.24 | 521.58 | 248.74 | 544.78 | 3.65 | 0.67 |
| 6 | 71.12 | 114.64 | 97.04 | 97.33 | 170.02 | 104.85 | 89.96 | 112.31 | 41.19 | 57.91 |
| 7 | 100.14 | 159.35 | 178.91 | 119.75 | 135.98 | LoS | 132.55 | 145.31 | 45.17 | 45.10 |
| 8 | 204.66 | 63.04 | 276.57 | 236.28 | 244.20 | 281.39 | 244.10 | 224.26 | 19.60 | 9.58 |
| **Average Difference** | | | | | | | | | **26.03** | **31.58** |
| **Standard Deviation** | | | | | | | | | **45.41** | **27.69** |

LoS = Loss of Signal

**Table 5.11: Comparison between Total Curved Lengths of Actual and Fitted Alignment of Highway 31**

| Curve | Actual Alignment | | | | Fitted Alignment | | | | $\%\lvert L_f - L_a \rvert$ |
|---|---|---|---|---|---|---|---|---|---|
| | $L_c$ | $L_{s1}$ | $L_{s2}$ | Total $L_a$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | Total $L_f$ | |
| **1** | 464.20 | 61.00 | 61.00 | 586.20 | 516.21 | 29.29 | 57.16 | 602.66 | 2.81 |
| **2** | 474.00 | 61.00 | 61.00 | 596.00 | 451.90 | 112.78 | 154.23 | 718.90 | 20.62 |
| **3** | 162.40 | 61.00 | 61.00 | 284.40 | 202.27 | 63.42 | 80.23 | 345.92 | 21.63 |
| **4** | 170.00 | 61.00 | 61.00 | 292.00 | 232.07 | 67.90 | 35.92 | 335.89 | 15.03 |
| **5** | 341.25 | 61.00 | 61.00 | 463.25 | 301.07 | 99.31 | 82.51 | 482.88 | 4.24 |
| **6** | 377.33 | 61.00 | 61.00 | 499.33 | 417.31 | 65.86 | 41.38 | 524.55 | 5.05 |
| **7** | 783.64 | 61.00 | 61.00 | 905.64 | 555.49 | 121.96 | 179.32 | 856.77 | **5.40** |
| **8** | 472.38 | 61.00 | 61.00 | 594.38 | 520.48 | 107.16 | 80.77 | 708.40 | 19.18 |
| **9** | 527.00 | 61.00 | 61.00 | 649.00 | 545.08 | 83.08 | 134.93 | 763.09 | 17.58 |
| **Average** | | | | | | | | | **12.39** |
| **Standard Deviation** | | | | | | | | | **7.86** |

$L_a$ = Actual length  $L_f$ = Fitted length

**Table 5.12: Comparison between Total Curved Lengths of Actual and Fitted Alignment of Highway 41**

| Curve | Actual Alignment | | | | Fitted Alignment | | | | % $\lvert L_f - L_a \rvert$ |
|---|---|---|---|---|---|---|---|---|---|
| | $L_c$ | $L_{s1}$ | $L_{s2}$ | Total $L_a$ | $L_c$ | $L_{s1}$ | $L_{s2}$ | Total $L_f$ | |
| **1** | 159.79 | 61.00 | 61.00 | 281.79 | 105.75 | 91.61 | 77.40 | 274.76 | 2.49 |
| **2** | 110.91 | 61.00 | 61.00 | 232.91 | 243.58 | 192.51 | 176.39 | 612.48 | 162.97 |
| **3** | 430.66 | 45.73 | 45.73 | 522.12 | 125.24 | 93.86 | 87.28 | 306.37 | 41.32 |
| **4** | 208.79 | 45.73 | 45.73 | 300.25 | 262.42 | 210.59 | 202.96 | 675.98 | 125.14 |
| **5** | 541.13 | 45.73 | 45.73 | 632.59 | 64.77 | 56.75 | 50.11 | 171.63 | 72.87 |
| **6** | 71.12 | 45.73 | 45.73 | 162.58 | 69.45 | 69.45 | 56.83 | 195.73 | 20.39 |
| **7** | 100.14 | 45.73 | 45.73 | 191.60 | 147.95 | 121.26 | 96.69 | 365.89 | 90.97 |
| **8** | 204.66 | 45.73 | 45.73 | 296.12 | 2.78 | 2.78 | 2.78 | 8.34 | 97.19 |
| **Average** | | | | | | | | | **76.67** |
| **Standard Deviation** | | | | | | | | | **53.93** |

$L_a$ = Actual length $L_f$ = Fitted length

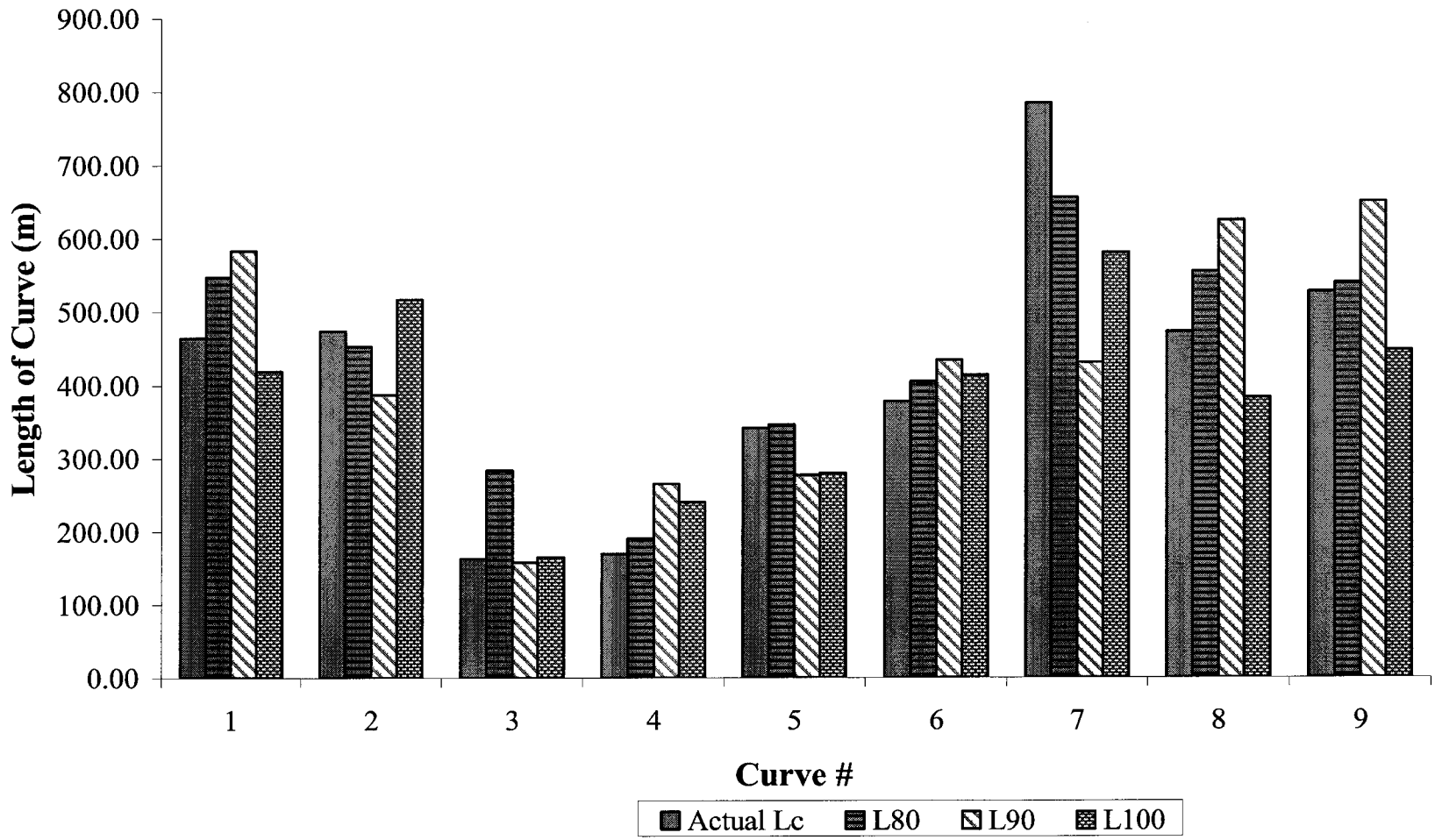



**Figure 5.5: Effect of Speed on Curve Length.**

Figure 5.6 shows the points where lateral displacement between the observed and actual alignment was measured. The position of these points is adjusted such that Point 5 is the mid-point of curve, Points 4, 6 are the start and end of curve, Points 3 and 7 are on the transition curves, and Points 1, 2 and 8, 9 are on the entry and exit tangents respectively. Spacing between all the points except those on the circular curve is 50 m. Figure 5.7 shows the position of both actual and observed alignments for a segment on Highway 31. All six fitted alignments were located on the western side of the actual alignment. However, the fitted alignments were still within the range of accuracy of the digitized maps of $\pm 10$ meters (Appendix A). Furthermore, the accuracy of the fitted alignments is evident through the following:

- For each travel direction, all the three observed alignments were quite close to each other. A slight variation can be attributed to difference in speed adopted for each observed alignment.
- If ideal conditions are assumed that the vehicle had travelled exactly in the center of each lane (3.35 meters average width), the lateral distance between the centerline of observed alignments should be 3.35 meters. However, in a real world, the vehicle does not follow exactly the centerline of the lane, and therefore the lateral distance between the observed alignments (north and south) varies between 3 and 4 meters (Table 5.13) for Highway 31.

Lower values for the lateral distance between observed alignments show that the vehicle moves close to the centerline whereas higher values indicate that the vehicle moves away from the centerline. The first half of Curve 5 indicates a significantly high

displacement. Similarly, the standard deviation for Curve 5 is quite high compared to those obtained for rest of the curves.

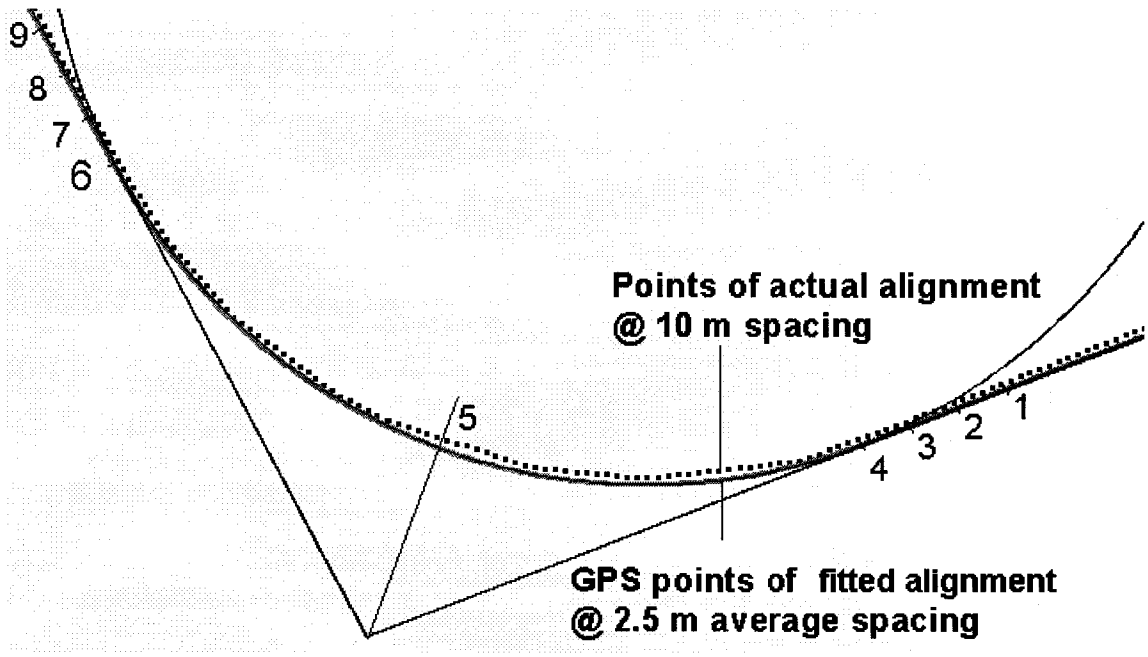



**Figure 5.6: Points Location for Measuring Lateral Displacement.**

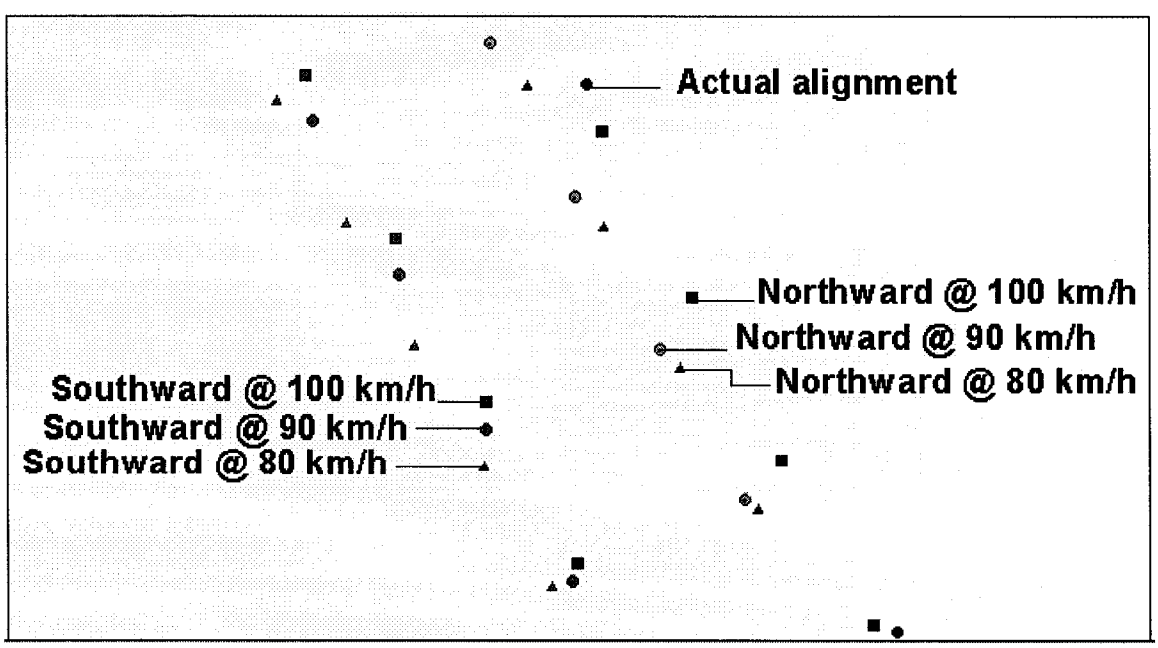



**Figure 5.7: A Glimpse of Enlarged View Showing Actual and Observed Highway 31.**

**Table 5.13: Average Lateral Distance between Observed Northward and Southward Alignments of Highway 31**

| Curvee | Average Lateral distance (m) at point: | | | | | | | | | Averagee | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| C1 | 3.83 | 3.49 | 4.02 | 3.85 | 2.56 | 3.18 | 3.01 | 3.53 | 3.78 | 3.47 | 0.47 |
| C2 | 3.81 | 3.61 | 3.46 | 3.42 | 3.51 | 4.28 | 4.56 | 4.95 | 4.82 | 4.05 | 0.61 |
| C3 | 4.28 | 3.90 | 4.08 | 3.47 | 3.81 | 3.46 | 3.22 | 3.74 | 3.74 | 3.74 | 0.33 |
| C4 | 3.21 | 3.22 | 3.43 | 3.32 | 3.42 | 2.81 | 2.86 | 3.13 | 3.29 | 3.19 | 0.22 |
| C5 | 8.73 | 7.99 | 7.43 | 7.59 | 5.14 | 4.15 | 3.46 | 3.64 | 3.45 | 5.73 | 2.18 |
| C6 | 3.51 | 3.82 | 3.88 | 3.68 | 3.83 | 3.87 | 3.42 | 3.39 | 3.37 | 3.64 | 0.22 |
| C7 | 3.32 | 3.24 | 3.22 | 3.22 | 3.26 | 3.27 | 3.82 | 3.83 | 3.95 | 3.46 | 0.31 |
| C8 | 3.27 | 3.19 | 3.67 | 3.37 | 6.21 | 3.13 | 2.85 | 3.09 | 3.14 | 3.55 | 1.02 |
| C9 | 2.81 | 2.69 | 3.04 | 3.58 | 3.51 | 4.16 | 4.28 | 4.36 | 4.21 | 3.63 | 0.66 |
| **Ave.** | 4.09 | 3.90 | 4.03 | 3.95 | 3.92 | 3.59 | 3.50 | 3.74 | 3.75 | **3.83** | **0.67** |
| **S.D.** | 1.79 | 1.57 | 1.33 | 1.38 | 1.10 | 0.54 | 0.61 | 0.60 | 0.53 | | **1.05** |

Due to the hilly topography of Highway 41 surrounded by forest, the fitted alignment is observed to be at quite larger distance on the eastern side from the actual alignment. Table 5.14 presents the lateral displacement of fitted alignment of Highway 41, while traveling towards south at 90 km/h, which was found the closest to the actual alignment. Starting from 17.45 m average displacement at Point 1, it decreases to 14.73 m at point 5, the middle of curve (15.59% decrease relative to Point 1). After the mid point of the curve, displacement increased to 18.41 m maximum at Point 8. However, after Point 8, displacement is reduced to 14.80 m on the next Point (9). In this alignment, only Curve 7 presents better results with the exception of Point 8 where due to complete

loss of signal no data could be observed. Positions of points used for measurement of lateral displacement of Highway 41 are shown in Figure 5.8 for a sample curve C6. The figure also presents both actual and fitted alignments. Results of Highway 41 give an idea of tracking relative alignment of a highway in unfavourable environment for GPS survey.

**Table 5.14: Lateral Distance between Actual and Observed Southward Alignment of Highway 41**

| | Lateral distance (m) at point: | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Curve** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **Average** | **S.D** |
| **C1** | 56.81 | 48.22 | 39.60 | 29.49 | 15.45 | 11.26 | 13.92 | 18.52 | 21.76 | 28.34 | 16.35 |
| **C2** | 11.26 | 13.92 | 18.52 | 21.76 | 20.52 | 15.37 | 13.23 | 10.58 | 8.64 | 14.87 | 4.57 |
| **C3** | 16.31 | 16.06 | 15.38 | 16.32 | 7.06 | 22.04 | 18.50 | 17.42 | 14.94 | 16.00 | 3.97 |
| **C4** | 6.16 | 5.08 | 4.35 | 4.03 | 6.76 | 17.91 | 18.24 | 17.90 | 6.10 | 9.61 | 6.36 |
| **C5** | 6.62 | 6.88 | 6.21 | 5.86 | 27.43 | 25.68 | 24.87 | 22.64 | 21.66 | 16.43 | 9.67 |
| **C6** | 24.87 | 22.64 | 21.66 | 19.96 | 18.42 | 15.95 | 15.49 | 15.77 | 14.13 | 18.77 | 3.73 |
| **C7** | 9.64 | 8.33 | 8.21 | 8.01 | 6.83 | 12.47 | 8.52 | LoS | 7.58 | 8.70 | 1.72 |
| **C8** | 7.92 | 7.37 | 5.84 | 6.58 | 15.37 | 19.59 | 25.58 | 26.03 | 23.62 | 15.32 | 8.60 |
| **Average** | 17.45 | 16.06 | 14.97 | 14.00 | 14.73 | 17.53 | 17.29 | 18.41 | 14.80 | **16.00** | **6.87** |
| **S.D.** | 17.07 | 14.25 | 11.85 | 9.24 | 7.50 | 4.83 | 5.81 | 4.92 | 6.96 | | **9.16** |

Table 5.15 summarizes the average lateral displacement measured between the actual and observed centerlines of Highway 31 at three different speeds (80, 90, and 100 km/h) for nine curves of different radii and lengths. This table also distinguishes between the behaviour of left-turn and right-turn curves for the same driver at the same speed traveling towards south. With a couple of exceptional cases, lateral displacement is generally constant along the entry tangents for all the three speeds. Passing through the entry side transition curve, a slight increase in lateral displacement for all three speeds indicates movement towards edge of the road. Then along the curve, from start (Point 4)

to middle of the curve (Point 5), this displacement increases further with the average increase for the displacement at Points 2 to 5 relative to Point 1 on all nine curves being 10.58%. The average displacement for all nine curves reached a maximum of 6.99 m at Point 5 at the middle of curve. After the curve mid point (Point 5), the displacement is reduced to an average of 6.21 m at Point 6 at the end of curve (1.73% decrease relative to Point 1). Along the exit transition curve and exit tangent, the displacements continue to decrease.

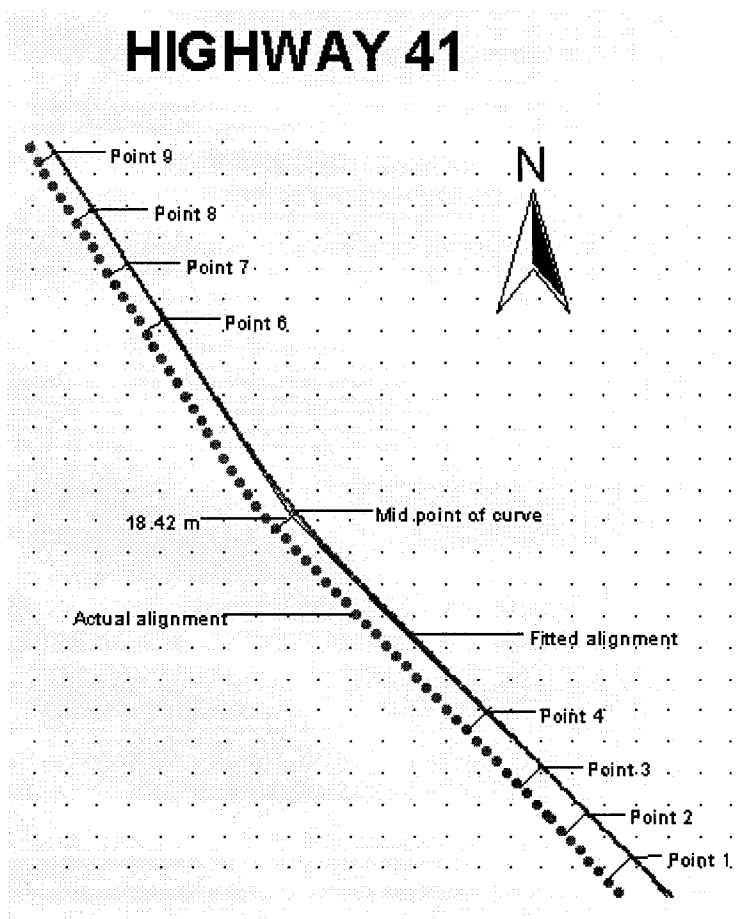



**Figure 5.8: Enlarged View of Curve 6, Highway 41, Measuring Lateral Displacement.**

**Table 5.15: Effect of Curve Position on Lateral Displacement for Southward Observed Alignment**

| Position | Average Lateral displacement (m) at speed: | | | | | |
|---|---|---|---|---|---|---|
| | 80 km/h | | 90 km/h | | 100 km/h | |
| | LT curve | RT curve | LT curve | RT curve | LT curve | RT curve |
| 1 | 5.41 | 7.08 | 5.36 | 7.18 | 5.26 | 7.62 |
| 2 | 5.42 | 7.03 | 5.38 | 7.06 | 5.30 | 6.85 |
| 3 | 5.43 | 6.77 | 5.44 | 6.76 | 5.41 | 6.56 |
| 4 | 6.03 | 6.59 | 5.73 | 6.50 | 5.56 | 6.64 |
| 5 | 6.51 | 7.43 | 6.53 | 7.33 | 6.70 | 7.43 |
| 6 | 5.50 | 6.85 | 5.46 | 6.90 | 5.50 | 7.03 |
| 7 | 5.10 | 6.39 | 4.94 | 6.46 | 4.84 | 6.55 |
| 8 | 4.85 | 6.61 | 4.76 | 6.31 | 4.80 | 6.47 |
| 9 | 4.87 | 6.02 | 5.16 | 5.68 | 5.12 | 6.02 |
| **Average** | 5.46 | 6.75 | 5.42 | 6.69 | 5.39 | 6.80 |
| **S.D** | 0.54 | 0.41 | 0.51 | 0.51 | 0.56 | 0.50 |

LT = Left Turn; RT = Right Turn

Figure 5.9 shows the driver's behaviour on left-turn and right-turn curves of Highway 31. The Figure compares average lateral displacements of all nine curves for the vehicle travelling at 80 km/h towards south. It can be observed that lateral displacement on right-turn curves is higher than for left-turn curves. This means that on right-turn curves, the vehicle moves away from the centerline, and on left-turn curves, the vehicle

moves towards centerline. This type of driver's behaviour is in accordance with previous studies, even though this study was based on a control driver. The figure also indicates the driver behaviour along various points on the curve, with the maximum displacement at the middle of the curve.

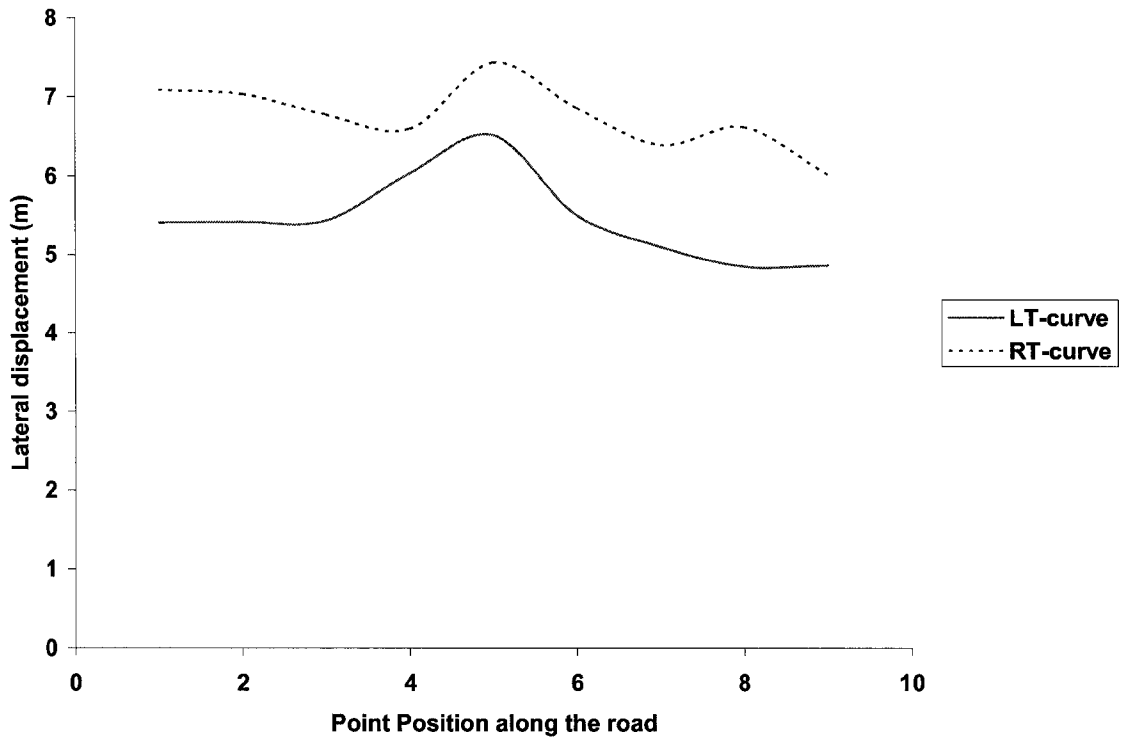



**Figure 5.9: Comparison of Average Lateral Displacement on Left Turn and Right Turn Curves, while Traveling @ 80 km/h towards South.**

# Chapter 6

# FITTING THE VERTICAL ALIGNMENT

## 6.1 Overview

The vertical alignment, consisting of vertical curves and grades, is an important design feature that affects highway safety, economy and aesthetics. Vertical alignment is usually presented in a profile, which is a graph with elevation as its vertical axis, and distance measured in stations along the centerline or other horizontal reference line of the facility as its horizontal axis. In the current study, the $x$, $y$ coordinates of the collected GPS data were converted to stations and the $z$ coordinates were used as elevations. Both the station and the elevation data for the actual and observed alignment were analysed using the software "VAFIT" (Easa et al., 1998). The graphical comparison between actual and observed profiles gave encouraging results.

## 6.2 VAFIT

In North America, the highway vertical alignment is developed as a series of straight segments (tangents) and parabolic curves; however, many existing highways were originally small roads that were built without following specific engineering standards (Easa et al., 1998). The information on the vertical alignment of existing old highways, such as grades and vertical curves, is often not available. Field measurements are carried out to establish the existing, as-built highway profile in the form of elevations of the highway centerline at the predefined stations as well as at the irregular locations of the profile. The straight lines and parabolic curves are then visually fitted to the profile.

This method, however, is time-consuming, laborious and may not be sufficiently accurate (Easa, 1999).

To overcome the limitation of the visual method, an analytical method was developed by Easa et al. (1998) to establish the vertical alignment of a highway using profile field data. The method was then coded into a computer program, namely "VAFIT" that determines the straight and curved segments from the available data and estimates the instantaneous grades. It examines the slopes of subsequent data points with respect to an initial point, and based on the trend of the slopes, determines whether the segment is tangent, a crest curve, or a sag curve. After identifying shape of the segments, it fits the tangent segment as straight lines and curved segments (crest and sag) using clamped cubic spline functions. Results of the analysis are displayed as text and as a script file that can be used to draw the alignment in AutoCAD drawing packages (Easa et al., 1998):

Previously this software was used to analyse data collected through traditional surveying technique. Such measurements included stations and elevations. In the current study, the observed GPS data were first converted to stations and elevations and then analysed using VAFIT.

## 6.3 Data Format

As discussed earlier, the collected GPS data contained four dimensions, i.e., $x$, $y$, $z$ and time. To find vertical alignment of the highway, the first three dimensions ($x$, $y$, and $z$) were used. First, the $x$, $y$ coordinates of successive points were converted to horizontal segments of length given by the relationship:

$$L = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2}$$

Where $L$ is length of each segment, $(x_1, y_1)$ and $(x_2, y_2)$ are the coordinates of first and second point.

The stations were determined by accumulating the lengths of successive segments along the highway alignment. The station and the elevation ($z$) data were then tabulated into a specific format with stations in first column and elevations in second column. The total number of points to be analysed were mentioned in the top most cell of the spreadsheet. Both stations and elevations were measured in meters.

## 6.4 Data Refinement

The GPS data were collected at 0.1 second interval, driving at 80, 90, and 100 km/h speed in both directions (north and south) of the two highways (Highway 31 and 41). As discussed earlier, the raw GPS data were then analysed in the GPS software (SKI Pro) for removal of ambiguities. Now if the speed remains constant, the spacing between two consecutive points (or length of each segment) should be 2.22 m, 2.50 m and 2.78 m for 80, 90 and 100 km/h respectively. However, practically the spacing between consecutive points varied due to some variation in the vehicle speed. Some points were found very close to each other, while a few were overlapping (having the same stations). The overlapped points showing zero length of the segment were causing error during analysis by VAFIT, and were therefore deleted from the database. Similarly, the points

having spacing less than 0.1 m were also removed to minimize the work load and save time.

To compare same segment in both actual and observed profiles a highway section was taken from the mid point of the first horizontal curve to the mid point of the last horizontal curve. For this purpose, each file of the observed horizontal alignment was opened in ArcView and the required points were selected graphically. Then from the attribute table the highlighted point identification of each selected Point was noted. Now the same file was opened in Microsoft Excel and the data before the mid point of first curve and after the mid point of the last curve were deleted. The left over length was optimised to the length of actual profile. Similarly, for actual profile data, the desired points were noted from the available drawings, and the same equal length of segment was selected from the database. Since the actual profile data were measured from south to north, hence only the northward observed profiles were analysed and compared with actual profiles. Some of the actual profile data had been measured in British system of units, which were converted to the metric units. Moreover, the first station for both types of data (actual and observed) was converted to 00 (zero) to have a common origin, and accordingly the remaining stations were calculated.

## 6.5 Data Analysis

Each file of the refined GPS data was saved in "delimited text" file format. Three files for each highway were developed, representing GPS data collected at 80, 90 and 100 km/h while driving towards north. Tables 6.1 and 6.2 describe the length, number of points, and the minimum and maximum elevation for Highway 31 and 41, respectively.

Each table provides a comparison between the actual profile and the observed three profiles. The number of points for the observed profiles is significantly high due to small spacing (caused by small observation interval) between them. Because VAFIT was unable to analyse a file containing more than 2000 points, the observed profile data were divided into sub-files, each containing 2000 or less number of points.

It can be observed in Table 6.1, that the minimum and maximum elevations of observed profiles for Highway 31 are lower than the actual profile by an average value of 31.76 m (43.27%) and 32.12 m (37.09%) respectively. However, all three observed profiles have almost the same elevations, which indicate that speed of the vehicle has not affected the GPS elevation data. On other hand, the elevations of the observed profiles for Highway 41 are not only different from the actual profile (18.61% and 11.67% lower than the actual minimum and maximum elevation, respectively) but also contain significant difference from each other (Table 6.2). As described in Chapter 5, this variation in observed profiles of Highway 41 may be attributed to its topography causing blockage of GPS signals and deteriorating the quality of data.

**Table 6.1: General Characteristics of Actual and Observed Profiles for Highway 31**

| Profile | Length (m) | No. of Points | Min. Elevation (m) | Max. Elevation (m) |
|---|---|---|---|---|
| Actual | 14,730 | 1,473 | 73.40 | 86.60 |
| 31N80 | 14,732.05 | 6,118 | 41.64 | 54.49 |
| 31N90 | 14,734.72 | 6,061 | 41.63 | 54.46 |
| 31N100 | 14,732.31 | 7,822 | 41.64 | 54.49 |

**Table 6.2: General Characteristics of Actual and Observed Profiles for Highway 41**

| Profile | Length (m) | No. of Points | Min. Elevation (m) | Max. Elevation (m) |
|---|---|---|---|---|
| Actual | 8,472.93 | 416 | 196.34 | 278.8 |
| 41N80 | 8,448.21 | 3,928 | 159.99 | 240.47 |
| 41N90 | 8,472.13 | 3,629 | 158.83 | 247.60 |
| 41N100 | 8,472.87 | 3,534 | 160.59 | 250.75 |

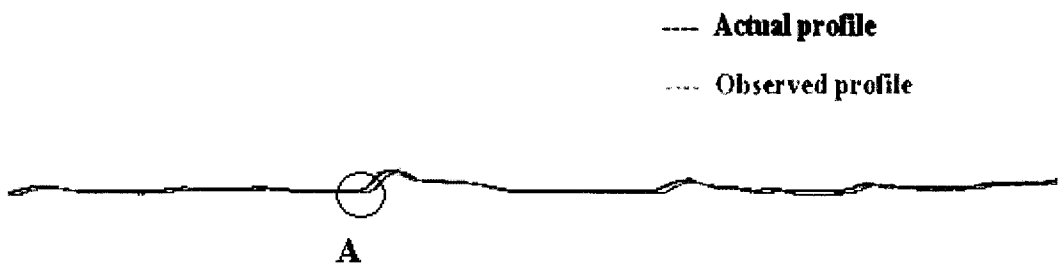
After analysis by VAFIT, each script file was converted to its graphical form using AutoCAD (version 2004). In the same drawing, first the script for actual profile was run. Then script for each sub-file of one observed profile was run. According to the difference in elevations shown in Tables 6.1 and 6.2, the observed profiles were plotted below the actual profiles (Figures 6.1 and 6.2).

As shown in Figures 6.1 and 6.2, the observed profiles collected at three different speeds are about 30 m lower than the actual profile, but this difference in elevation remains constant throughout the section. In other words the observed profiles are almost parallel to the actual profile. This indicates that although the absolute position of the observed profile is displaced, its relative positioning is very close to the actual one. It should also be noted that the actual elevation of the base station was not known. Hence, the elevations collected by the rover could not be corrected for absolute vertical positioning. Figure 6.1 represents graphical results of VAFIT for actual and one fitted profile (80 km/h) of Highway 31. Part (c) of Figure 6.1 indicates that difference between actual and fitted profiles increases along vertical curve compared to tangent. Figure 6.2 compares actual elevations with all the three observed elevations without fitting the geometry of the features. This figure was developed in Microsoft Excel using directly the

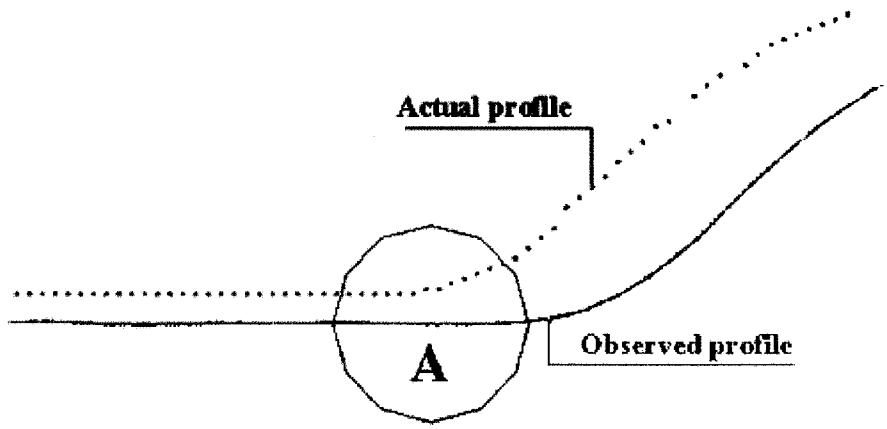
station and elevation data for finding values of station and elevation of any point along the profile.

Figure 6.3 compares actual elevations with elevations observed at three different speeds for Highway 41. Again, the observed profiles are around 35 m lower than actual profile and run somewhat parallel to the actual profile. However, in this case the profile is not as smooth as was for Highway 31. This is due to the loss of GPS signals during data observation. Also, several small sections are missing along the alignment, due to non-availability of data. However, the general trend of the observed profiles is relatively parallel to the actual profile. An extract from text output of VAFIT for actual profile of Highway 41 is presented in Table 6.3. In this table, $g_1$ and $g_2$ are the gradients of first and second segments respectively, and $K_{av}$ is the curve length per 1 percent change in grade. Grade at any point of the actual profile shown in Figure 6.3 can be easily found in this table.

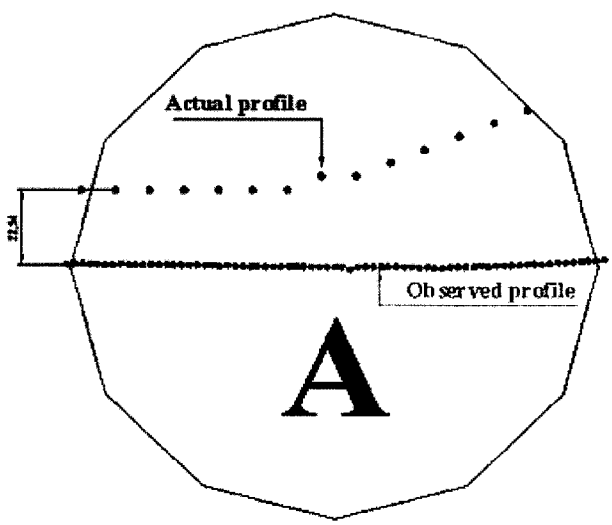
As mentioned in Chapter 3, the GPS does not give altitudes as accurately as it gives the horizontal coordinates (Czerniak 1998). Results of the current study comply with this fact. However, it can be observed from the current study that for tracking relative vertical alignment of the highway, the GPS provides significantly accurate data. Results can further be improved if the environment of the highway is open to avoid blockage of GPS satellite signals. The procedure can be used for developing vertical alignment of the existing highways. For this purpose, it is advisable to provide local coordinates instead of absolute coordinates.




(a) Highway 31.




(b) Zooming a Small Segment of Highway 31.




**Measured vertical difference = 22.54 m**

(c) Enlarged View of Section A

**Figure 6.1: Output of VAFIT, Actual vs. Fitted Profile of Highway 31 (80 km/h file)**

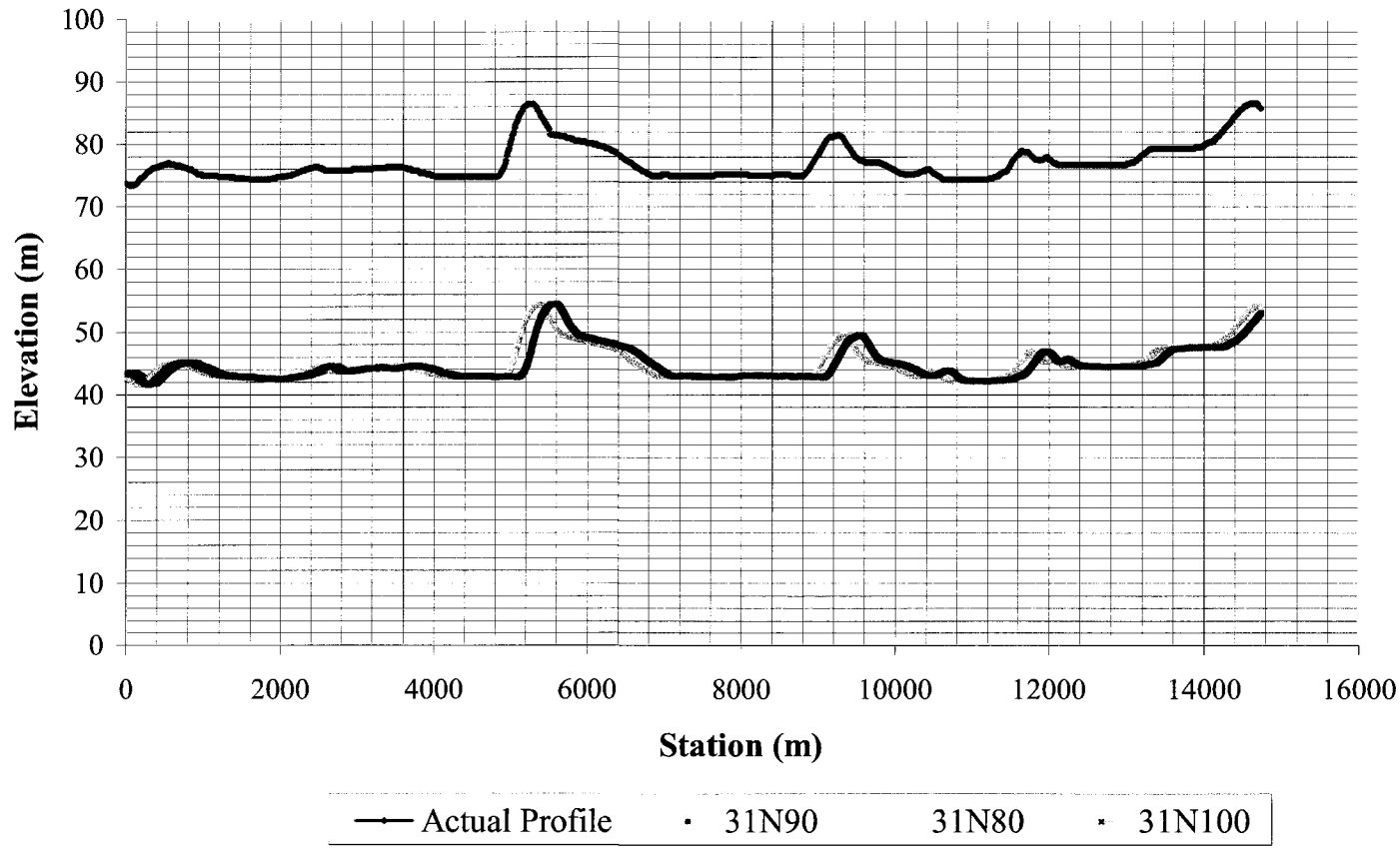



**Figure 6.2**: **Actual vs. all Observed Profiles of Highway 31.**

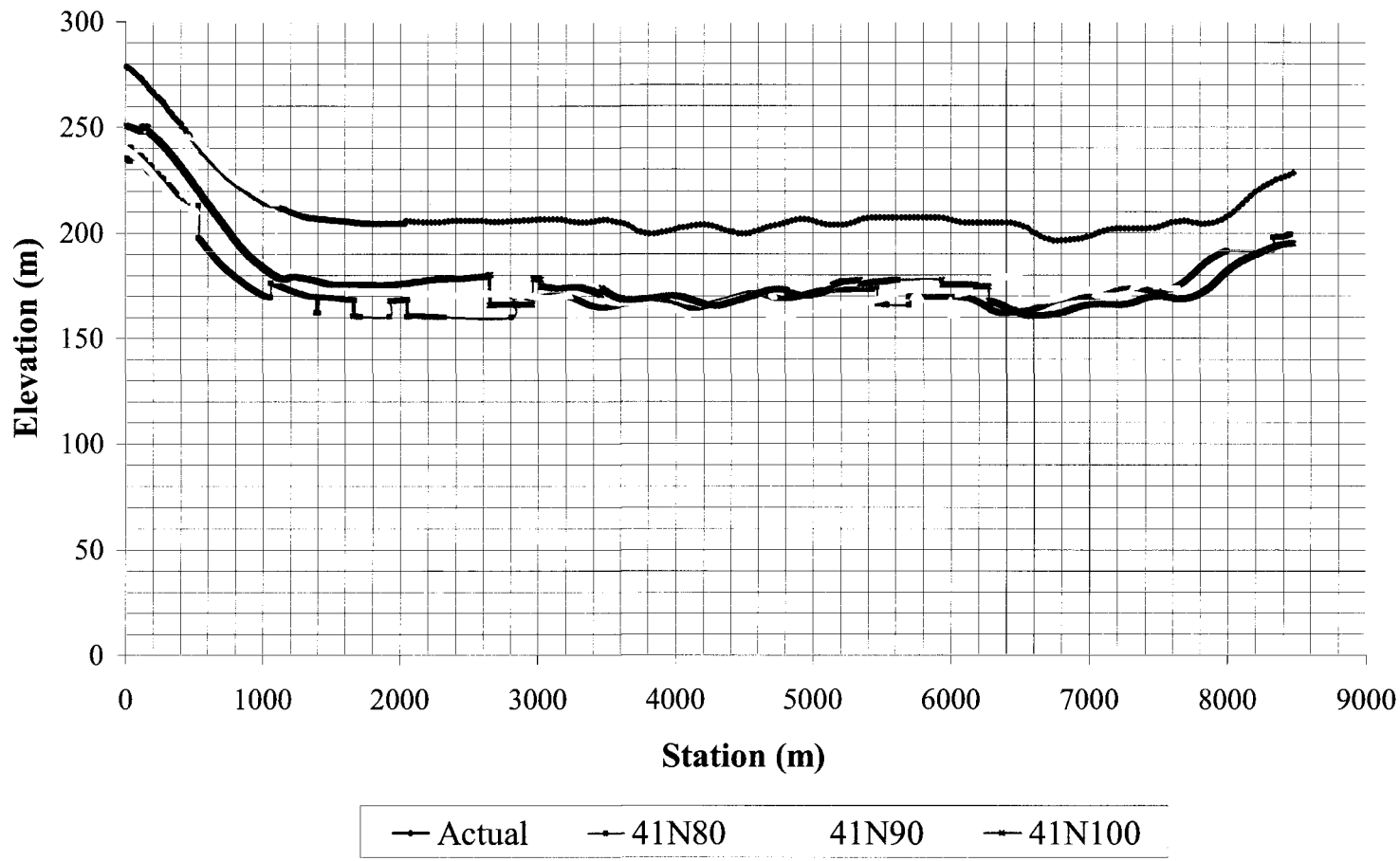



**Figure 6.3**: **Actual vs. all Observed Profiles of Highway 41.**

**Table 6.3: Results of VAFIT for Actual Profile of Highway 41**

| Station (m) | | Length (m) | Shape | Gradient | | $K_{av}$ |
|---|---|---|---|---|---|---|
| From | To | | | $g_1$ (%) | $g_2$ (%) | |
| 0.00 | 560.00 | 560.00 | straight | --- | --- | --- |
| 560.00 | 600.00 | 40.00 | sag | -7.49 | -5.22 | -17.62 |
| 600.00 | 980.00 | 380.00 | straight | --- | --- | --- |
| 980.00 | 1020.00 | 40.00 | sag | -5.22 | -0.23 | -8.01 |
| 1020.00 | 3198.54 | 2178.54 | straight | --- | --- | --- |
| 3198.54 | 3442.44 | 243.90 | sag | -0.23 | -2.08 | 132.00 |
| 3442.44 | 3747.32 | 304.88 | straight | --- | --- | --- |
| 3747.32 | 3899.76 | 152.44 | sag | -2.08 | 1.40 | -43.90 |
| 3899.76 | 4082.68 | 182.93 | straight | --- | --- | --- |
| 4082.68 | 4296.98 | 214.30 | crest | 1.40 | 0.38 | 209.65 |
| 4296.98 | 4722.93 | 425.95 | straight | --- | --- | --- |
| 4722.93 | 4844.88 | 121.95 | sag | 0.38 | 2.19 | -67.27 |
| 4844.88 | 5027.81 | 182.93 | crest | 2.19 | -2.19 | 41.73 |
| 5027.81 | 5149.76 | 121.95 | sag | -2.19 | 0.52 | -44.92 |
| 5149.76 | 5850.98 | 701.22 | straight | --- | --- | --- |
| 5850.98 | 5972.93 | 121.95 | crest | 0.52 | -1.67 | 55.70 |
| 5972.93 | 6003.42 | 30.49 | straight | --- | --- | --- |
| 6003.42 | 6125.37 | 121.95 | sag | -1.67 | -0.31 | -89.62 |
| 6125.37 | 7436.34 | 1310.98 | straight | --- | --- | --- |
| 7436.34 | 7558.29 | 121.95 | sag | -0.31 | 0.14 | -274.66 |
| 7558.29 | 7771.71 | 213.42 | straight | --- | --- | --- |
| 7771.71 | 7924.15 | 152.44 | sag | 0.14 | 4.55 | -34.58 |
| 7924.15 | 8472.93 | 548.78 | straight | --- | --- | --- |

# Chapter 7

# CONCLUSIONS AND RECOMMENDATIONS

## 7.1 Overview

Highway professionals strive to improve highway alignment design and improve highway safety. One of the major aspects of their research studies is to find the exact path of the vehicle while negotiating a curve. In this context, work has been in progress to find suitable methods for tracking vehicle trajectory. The current study is an effort in this direction, wherein the available advanced technologies, GPS and GIS, have been employed to find vehicle path. The procedure is simple and efficient and can also be used for developing alignments of the existing highways.

## 7.2 Conclusions

This thesis presented the efforts made for the development of a procedure to track a vehicle path as it negotiates a segment of a rural highway using differential GPS surveying. The procedure was applied for tracking both horizontal as well as vertical alignment of the highways. For tracking horizontal alignment of the highway, an algorithm was worked out and developed into an extension for use in ArcView GIS, to fit the collected GPS data into a horizontal alignment composed of tangents and simple horizontal curves with or without spiral transitions. These tools can be used to examine the behaviour of a driver under normal driving conditions (driver is instructed to drive as they normally do) or to extract the highway horizontal alignment under controlled driving

conditions (driver is instructed to track the highway centerline as accurately as possible). The procedure and algorithm have proven to be relatively inexpensive, simple, precise, and automated. Integration into a GIS environment has enabled the utilization of GIS capabilities including map and graphical displays and the ability to integrate the fitted alignment with available digital maps. For tracking vertical alignment, the GPS's three dimensional data were analysed using the available software.

An application was presented using two different sections of rural highways in eastern Ontario. One section was selected on Highway 31 that was relatively flat with an open environment, and involved nine different curves. The second section was taken from a hilly region of Highway 41, which contained relatively steep grades and was surrounded by forests. Eight curves on Highway 41 were considered for study. Data were collected by a GPS fitted vehicle with a driver tracking the lane centerline as accurately as possible at three different speeds and for the two directions of travel. The observed GPS data were refined using GPS software and then analysed for obtaining the highway alignment. The observed horizontal as well as vertical alignments were compared with the actual alignment and the following conclusions were made:

- The GPS data are in digital format that can be operated comfortably in different software packages; however, GIS is the best environment for GPS database, analysis and interpretation of results in both graphic as well as text shape.
- The GIS extension developed in this study can fit the scattered GPS points into proper horizontal alignment and determine various geometric features of the road, including tangent, transition, and circular curve.

- The fitted geometric features determined by the developed extension were reasonably close to their corresponding actual values.
- The observed geometric features represent the alignment followed by the vehicle, and thus can be used to provide an assessment of the driver's behaviour on various sections of a road.
- The fitted horizontal alignment was found very close to its actual position.
- Through the application, it was evident that the travel speed did not affect the quality of data or fitting.
- It was also found that a smaller observation interval would provide better alignment fitting.
- Using "VAFIT" the three dimensional GPS data can be easily transformed to vertical alignment of the highway; the method greatly reduces the time and effort required for vertical alignment preparation.
- The fitted vertical alignment has very close likeness with the actual profile but its absolute position is considerably different than the actual one.
- GPS data collected in open environment gave better results as compared to that in congested environment.
- Finally, this GPS-GIS-based method allows rapid determination of the lateral placement of a vehicle on every point of the highway alignment. Thus, conflict areas, caused by critical encroachment of the centerline or the edge lines, can also be easily detected.

## 7.3 Recommendations:

- Although the procedure developed in this study seems to be economical, a careful economic analysis needs to be carried out and compared with the methods previously used for the same purpose.
- The scope of the GIS extension developed in this study should be enhanced to include compound curves as well.
- The procedure should be repeated for the GPS data collected without using control station.
- To study driver's behaviour, data should be observed simultaneously for highway alignment as well as vehicle path, using the same base station. This procedure would also resolve the problem of error associated with the base map (as was in the digitized plans of Highways 31 and 41).

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# APPENDEX (A)

# VEHICLE'S LATERAL DISPLACEMENT

## LATERAL DISTANCE BETWEEN ACTUAL ALIGNMENT AND VEHICLE TRAJECTORY

**Table A.1: TRAVEL DIRECTION: SOUTH** **SPEED: 80 Km/h**

| | Lateral distance at point (meter) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Curve** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **Average** | **S.D** |
| **C1** | 4.43 | 4.42 | 4.84 | 6.06 | 2.12 | 2.95 | 2.31 | 2.91 | 3.40 | 3.72 | 1.30 |
| **C2** | 5.35 | 5.31 | 5.37 | 4.55 | 5.13 | 4.10 | 4.64 | 4.78 | 4.20 | 4.83 | 0.49 |
| **C3** | 6.13 | 6.40 | 6.25 | 4.71 | 8.29 | 11.00 | 7.93 | 7.80 | 6.72 | 7.25 | 1.79 |
| **C4** | 4.23 | 4.42 | 4.81 | 5.65 | 4.89 | 5.35 | 4.13 | 4.25 | 5.35 | 4.79 | 0.57 |
| **C5** | 8.48 | 8.26 | 8.67 | 9.13 | 12.70 | 10.47 | 10.15 | 10.94 | 9.51 | 9.81 | 1.42 |
| **C6** | 10.63 | 10.27 | 8.25 | 6.90 | 3.24 | 3.72 | 5.80 | 5.02 | 4.04 | 6.43 | 2.77 |
| **C7** | 5.88 | 5.52 | 5.22 | 6.20 | 9.42 | 5.07 | 5.21 | 4.58 | 4.23 | 5.70 | 1.52 |
| **C8** | 5.74 | 5.80 | 5.83 | 6.15 | 10.79 | 6.12 | 5.74 | 6.38 | 6.41 | 6.55 | 1.61 |
| **C9** | 6.17 | 6.42 | 6.33 | 7.73 | 6.60 | 7.49 | 6.42 | 5.79 | 5.68 | 6.51 | 0.69 |
| **Average** | 6.34 | 6.31 | 6.17 | 6.34 | 7.02 | 6.25 | 5.81 | 5.83 | 5.50 | **6.18** | **1.35** |
| **S.D** | *2.02* | *1.89* | *1.41* | *1.44* | *3.54* | *2.87* | *2.25* | *2.36* | *1.88* | | ***2.19*** |

**Table A.2: TRAVEL DIRECTION: NORTH** **SPEED: 80 Km/h**

| | Lateral distance at point (meter) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Curve** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **Average** | **S.D** |
| **C1** | 0.38 | 1.03 | 0.72 | 1.94 | 1.24 | 0.45 | 0.10 | 0.18 | 0.11 | 0.68 | 0.62 |
| **C2** | 1.25 | 1.58 | 1.46 | 0.55 | 1.75 | 0.00 | 0.21 | 0.10 | 0.00 | 0.77 | 0.73 |
| **C3** | 1.88 | 2.25 | 2.08 | 1.49 | 4.11 | 7.19 | 4.67 | 3.59 | 3.53 | 3.42 | 1.79 |
| **C4** | 0.85 | 1.31 | 1.50 | 2.11 | 1.85 | 2.52 | 1.40 | 1.31 | 2.27 | 1.68 | 0.54 |
| **C5** | 1.27 | 0.83 | 1.22 | 1.92 | 7.79 | 6.06 | 6.47 | 7.12 | 5.30 | 4.22 | 2.86 |
| **C6** | 6.73 | 5.96 | 3.77 | 2.62 | -0.15 | 0.00 | 2.24 | 1.14 | 0.31 | 2.51 | 2.54 |
| **C7** | 2.11 | 2.15 | 2.23 | 2.95 | 7.05 | 1.96 | 1.33 | 0.85 | 0.77 | 2.38 | 1.89 |
| **C8** | 2.87 | 2.70 | 2.36 | 2.88 | 4.28 | 3.06 | 2.67 | 2.76 | 3.32 | 2.99 | 0.55 |
| **C9** | 3.36 | 3.42 | 3.45 | 3.87 | 3.74 | 3.52 | 2.05 | 1.53 | 1.35 | 2.92 | 0.99 |
| **Average** | 2.30 | 2.36 | 2.09 | 2.26 | 3.52 | 2.75 | 2.35 | 2.06 | 1.88 | **2.40** | **1.39** |
| **S.D** | *1.91* | *1.58* | *1.01* | *0.96* | *2.65* | *2.56* | *2.07* | *2.21* | *1.85* | | ***1.87*** |

**Table A.3: TRAVEL DIRECTION: SOUTH** **SPEED: 90 Km/h**

| Curve | Lateral distance at point (meter) | | | | | | | | | Average | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| **C1** | 4.55 | 5.19 | 5.80 | 6.65 | 1.75 | 2.42 | 2.03 | 2.63 | 3.10 | 3.79 | 1.79 |
| **C2** | 5.11 | 5.38 | 5.16 | 3.90 | 4.75 | 3.60 | 4.03 | 4.33 | 4.47 | 4.53 | 0.62 |
| **C3** | 6.39 | 6.29 | 5.95 | 4.35 | 8.04 | 11.20 | 7.82 | 7.16 | 6.34 | 7.06 | 1.90 |
| **C4** | 4.47 | 5.05 | 5.47 | 5.77 | 4.75 | 5.72 | 4.70 | 4.72 | 6.10 | 5.19 | 0.58 |
| **C5** | 8.64 | 8.27 | 8.46 | 9.39 | 12.55 | 10.89 | 10.37 | 10.96 | 9.02 | 9.84 | 1.44 |
| **C6** | 10.48 | 9.85 | 7.96 | 6.24 | 4.07 | 3.72 | 5.93 | 4.74 | 3.77 | 6.31 | 2.58 |
| **C7** | 5.82 | 5.16 | 4.79 | 5.73 | 9.50 | 5.02 | 4.98 | 4.00 | 4.28 | 5.48 | 1.62 |
| **C8** | 5.83 | 5.71 | 5.61 | 5.88 | 10.22 | 6.29 | 6.14 | 6.07 | 6.18 | 6.44 | 1.44 |
| **C9** | 6.02 | 5.93 | 6.35 | 7.50 | 7.11 | 7.49 | 6.05 | 5.97 | 5.78 | 6.47 | 0.70 |
| **Average** | 6.37 | 6.31 | 6.17 | 6.16 | 6.97 | 6.26 | 5.78 | 5.62 | 5.45 | **6.12** | **1.41** |
| **S.D** | *1.98* | *1.65* | *1.24* | *1.63* | *3.44* | *3.11* | *2.36* | *2.40* | *1.78* | | ***2.18*** |

**Table A.4: TRAVEL DIRECTION: NORTH** **SPEED: 90 Km/h**

| Curve | Lateral distance at point (meter) | | | | | | | | | Average | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| **C1** | 0.73 | 1.53 | 1.61 | 2.26 | -1.48 | -0.72 | -0.72 | -0.76 | -0.37 | 0.23 | 1.32 |
| **C2** | 1.24 | 1.62 | 2.13 | 1.24 | 1.67 | -0.12 | -0.20 | -0.49 | -0.51 | 0.73 | 1.05 |
| **C3** | 1.81 | 2.58 | 2.11 | 1.48 | 4.97 | 8.12 | 4.62 | 3.66 | 2.49 | 3.54 | 2.11 |
| **C4** | 1.06 | 1.46 | 1.90 | 2.60 | 1.08 | 2.55 | 1.35 | 1.39 | 2.69 | 1.79 | 0.67 |
| **C5** | 0.81 | 0.42 | 1.39 | 2.06 | 7.52 | 6.70 | 7.06 | 7.68 | 6.46 | 4.46 | 3.17 |
| **C6** | 7.50 | 6.45 | 4.20 | 3.16 | -0.21 | -0.11 | 2.68 | 1.81 | 0.80 | 2.92 | 2.74 |
| **C7** | 2.46 | 1.93 | 1.47 | 2.26 | 6.11 | 1.31 | 0.85 | 0.04 | 0.05 | 1.83 | 1.82 |
| **C8** | 2.01 | 2.13 | 1.96 | 2.60 | 4.20 | 3.10 | 3.52 | 3.45 | 2.94 | 2.88 | 0.77 |
| **C9** | 3.15 | 3.21 | 2.74 | 3.82 | 2.90 | 3.10 | 1.95 | 1.50 | 1.34 | 2.63 | 0.85 |
| **Average** | 2.31 | 2.37 | 2.17 | 2.39 | 2.97 | 2.66 | 2.35 | 2.03 | 1.77 | **2.33** | **1.61** |
| **S.D** | *2.11* | *1.72* | *0.86* | *0.79* | *2.98* | *3.07* | *2.45* | *2.63* | *2.20* | | ***2.09*** |

**Note: Negative value shows that vehicle trajectory is on eastern side of the actual alignment.**

**Table A.5: TRAVEL DIRECTION: SOUTH** **SPEED: 100 Km/h**

| Curve | Lateral distance at point (meter) | | | | | | | | | Average | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| C1 | 4.13 | 4.18 | 4.33 | 5.12 | 1.95 | 2.74 | 2.77 | 3.33 | 3.85 | 3.60 | 0.98 |
| C2 | 5.01 | 5.16 | 5.13 | 4.13 | 4.86 | 4.53 | 4.55 | 4.73 | 4.63 | 4.75 | 0.33 |
| C3 | 5.90 | 5.92 | 6.03 | 5.26 | 8.71 | 11.20 | 7.54 | 6.95 | 6.52 | 7.11 | 1.85 |
| C4 | 4.05 | 4.54 | 5.17 | 5.66 | 4.63 | 5.40 | 4.28 | 4.65 | 6.05 | 4.94 | 0.67 |
| C5 | 11.82 | 8.68 | 8.70 | 10.15 | 12.76 | 10.79 | 10.17 | 10.82 | 9.13 | 10.34 | 1.39 |
| C6 | 10.56 | 9.94 | 7.91 | 6.56 | 3.82 | 3.67 | 5.68 | 4.93 | 4.14 | 6.36 | 2.60 |
| C7 | 5.58 | 5.48 | 5.03 | 5.48 | 10.18 | 4.66 | 4.75 | 4.21 | 4.28 | 5.52 | 1.82 |
| C8 | 5.70 | 5.53 | 5.85 | 6.13 | 9.90 | 6.77 | 6.57 | 6.34 | 6.44 | 6.58 | 1.31 |
| C9 | 6.41 | 6.02 | 6.30 | 6.95 | 7.12 | 7.41 | 5.76 | 5.61 | 5.53 | 6.35 | 0.69 |
| **Average** | 6.57 | 6.16 | 6.05 | 6.16 | 7.10 | 6.35 | 5.79 | 5.73 | 5.62 | **6.17** | **1.29** |
| **S.D** | *2.75* | *1.91* | *1.42* | *1.71* | *3.54* | *3.00* | *2.15* | *2.20* | *1.66* | | ***2.26*** |

**Table A.6: TRAVEL DIRECTION: NORTH** **SPEED: 100 Km/h**

| Curve | Lateral distance at point (meter) | | | | | | | | | Average | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| C1 | 0.50 | 0.77 | 0.58 | 2.09 | -1.62 | -1.16 | -1.30 | -1.13 | -0.74 | -0.22 | 1.25 |
| C2 | 1.56 | 1.81 | 1.69 | 0.52 | 0.80 | -0.50 | -0.47 | -0.63 | -0.65 | 0.46 | 1.05 |
| C3 | 1.90 | 2.09 | 1.81 | 0.94 | 4.54 | 7.72 | 4.33 | 3.43 | 2.35 | 3.23 | 2.07 |
| C4 | 1.21 | 1.58 | 1.76 | 2.40 | 1.09 | 2.98 | 1.78 | 1.53 | 2.66 | 1.89 | 0.65 |
| C5 | 0.66 | 0.00 | 0.92 | 1.91 | 7.29 | 6.93 | 6.78 | 6.99 | 5.56 | 4.12 | 3.15 |
| C6 | 6.91 | 6.20 | 4.51 | 2.87 | 0.00 | -0.40 | 2.23 | 1.58 | 0.73 | 2.74 | 2.63 |
| C7 | 2.74 | 2.37 | 1.68 | 2.53 | 6.17 | 1.68 | 1.31 | 0.41 | 0.12 | 2.11 | 1.77 |
| C8 | 2.57 | 2.63 | 1.95 | 2.56 | 3.79 | 3.64 | 3.70 | 3.31 | 3.36 | 3.06 | 0.65 |
| C9 | 3.66 | 3.68 | 3.67 | 3.75 | 3.66 | 3.28 | 1.39 | 1.26 | 1.48 | 2.87 | 1.13 |
| **Average** | 2.41 | 2.35 | 2.06 | 2.17 | 2.86 | 2.69 | 2.19 | 1.86 | 1.65 | **2.25** | **1.59** |
| **S.D** | *1.97* | *1.79* | *1.25* | *0.98* | *2.97* | *3.17* | *2.47* | *2.47* | *2.06* | | ***2.12*** |

**Table A.7: Lateral Distance between Northward and Southward Alignment Observed @ 80 km/h**

| Curve | Lateral distance (meters) at point | | | | | | | | | Average | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| **C1** | 4.05 | 3.39 | 4.12 | 4.12 | 0.88 | 2.50 | 2.21 | 2.73 | 3.29 | 3.03 | 1.08 |
| **C2** | 4.10 | 3.73 | 3.91 | 4.00 | 3.38 | 4.10 | 4.43 | 4.68 | 4.20 | 4.06 | 0.38 |
| **C3** | 4.25 | 4.15 | 4.17 | 3.22 | 4.18 | 3.81 | 3.26 | 4.21 | 3.19 | 3.83 | 0.47 |
| **C4** | 3.38 | 3.11 | 3.31 | 3.54 | 3.04 | 2.83 | 2.73 | 2.94 | 3.08 | 3.11 | 0.26 |
| **C5** | 7.21 | 7.43 | 7.45 | 7.21 | 4.91 | 4.41 | 3.68 | 3.82 | 4.21 | 5.59 | 1.68 |
| **C6** | 3.90 | 4.31 | 4.48 | 4.28 | 3.39 | 3.72 | 3.56 | 3.88 | 3.73 | 3.92 | 0.37 |
| **C7** | 3.77 | 3.37 | 2.99 | 3.25 | 2.37 | 3.11 | 3.88 | 3.73 | 3.46 | 3.33 | 0.47 |
| **C8** | 2.87 | 3.10 | 3.47 | 3.27 | 6.51 | 3.06 | 3.07 | 3.62 | 3.09 | 3.56 | 1.13 |
| **C9** | 2.81 | 3.00 | 2.88 | 3.86 | 2.86 | 3.97 | 4.37 | 4.26 | 4.33 | 3.59 | 0.69 |
| **Average** | 4.04 | 3.95 | 4.09 | 4.08 | 3.50 | 3.50 | 3.47 | 3.76 | 3.62 | **3.78** | **0.73** |
| **S.D** | *1.30* | *1.38* | *1.38* | *1.24* | *1.59* | *0.65* | *0.73* | *0.62* | *0.51* | | ***1.04*** |

**Table A.8: Lateral Distance between Northward & Southward Alignment Observed @ 90 km/h**

| Curve | Lateral distance (meters) at point | | | | | | | | | Average | S.D |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | |
| **C1** | 3.82 | 3.66 | 4.19 | 4.39 | 3.23 | 3.14 | 2.75 | 3.39 | 3.47 | 3.56 | 0.52 |
| **C2** | 3.87 | 3.76 | 3.03 | 2.66 | 3.08 | 3.72 | 4.23 | 4.82 | 4.98 | 3.79 | 0.79 |
| **C3** | 4.58 | 3.71 | 3.84 | 2.87 | 3.07 | 3.08 | 3.20 | 3.50 | 3.85 | 3.52 | 0.54 |
| **C4** | 3.41 | 3.59 | 3.57 | 3.17 | 3.67 | 3.17 | 3.35 | 3.33 | 3.41 | 3.41 | 0.18 |
| **C5** | 7.83 | 7.85 | 7.07 | 7.33 | 5.03 | 4.19 | 3.31 | 3.28 | 2.56 | 5.38 | 2.15 |
| **C6** | 2.98 | 3.40 | 3.76 | 3.08 | 4.28 | 3.83 | 3.25 | 2.93 | 2.97 | 3.39 | 0.47 |
| **C7** | 3.36 | 3.23 | 3.32 | 3.47 | 3.39 | 3.71 | 4.13 | 3.96 | 4.23 | 3.64 | 0.38 |
| **C8** | 3.82 | 3.58 | 3.65 | 3.28 | 6.02 | 3.19 | 2.62 | 2.62 | 3.24 | 3.56 | 1.01 |
| **C9** | 2.87 | 2.72 | 3.61 | 3.68 | 4.21 | 4.39 | 4.10 | 4.47 | 4.44 | 3.83 | 0.66 |
| **Average** | 4.06 | 3.94 | 4.00 | 3.77 | 4.00 | 3.60 | 3.44 | 3.59 | 3.68 | **3.79** | **0.74** |
| **S.D** | *1.51* | *1.50* | *1.19* | *1.43* | *1.00* | *0.49* | *0.59* | *0.71* | *0.76* | | ***1.02*** |

**Table A.9: Lateral Distance between Northward & Southward Alignment Observed @ 100 km/h**

| | Lateral distance (meters) at point | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Curve** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **Average** | **S.D** |
| **C1** | 3.63 | 3.41 | 3.75 | 3.03 | 3.57 | 3.90 | 4.07 | 4.46 | 4.59 | 3.82 | 0.50 |
| **C2** | 3.45 | 3.35 | 3.44 | 3.61 | 4.06 | 5.03 | 5.02 | 5.36 | 5.28 | 4.29 | 0.87 |
| **C3** | 4.00 | 3.83 | 4.22 | 4.32 | 4.17 | 3.48 | 3.21 | 3.52 | 4.17 | 3.88 | 0.39 |
| **C4** | 2.84 | 2.96 | 3.41 | 3.26 | 3.54 | 2.42 | 2.50 | 3.12 | 3.39 | 3.05 | 0.40 |
| **C5** | 11.16 | 8.68 | 7.78 | 8.24 | 5.47 | 3.86 | 3.39 | 3.83 | 3.57 | 6.22 | 2.82 |
| **C6** | 3.65 | 3.74 | 3.40 | 3.69 | 3.82 | 4.07 | 3.45 | 3.35 | 3.41 | 3.62 | 0.24 |
| **C7** | 2.84 | 3.11 | 3.35 | 2.95 | 4.01 | 2.98 | 3.44 | 3.80 | 4.16 | 3.40 | 0.49 |
| **C8** | 3.13 | 2.90 | 3.90 | 3.57 | 6.11 | 3.13 | 2.87 | 3.03 | 3.08 | 3.52 | 1.02 |
| **C9** | 2.75 | 2.34 | 2.63 | 3.20 | 3.46 | 4.13 | 4.37 | 4.35 | 3.85 | 3.45 | 0.77 |
| **Average** | 4.16 | 3.81 | 3.99 | 3.99 | 4.25 | 3.67 | 3.59 | 3.87 | 3.94 | **3.92** | **0.83** |
| **S.D** | *2.66* | *1.88* | *1.49* | *1.65* | *0.92* | *0.76* | *0.78* | *0.75* | *0.69* | | ***1.29*** |