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The moving observer method revisited

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Abstract
The present paper proposes methodological and technological improvements in order to enrich the traditional moving observer method by direct measurement of the speed and length of each vehicle by the observer, while the former moves in one lane and the latter in an adjacent lane. This is realized using two laser detectors located on the observer’s vehicle and acting as a mobile double presence-type detector. In addition, the observer’s vehicle is equipped with a DGPS-based system that provides absolute positioning of all events during these meetings with the recorded vehicles (activation and deactivation of detectors) as well as their timing. These capabilities provide sufficient information for determining the prevailing speed distribution in non-congested conditions for short time periods of measurements (i.e. few minutes). The distribution contributes to reliable estimates of average flow and mean space speed for a relatively long highway section. The method proves its value, when fixed detector and moving measurements are combined; then the exact space speed distribution could be determined, and the exact mean space speed could be calculated. Finally, the method has the capability to follow the traffic stream characteristics along successive sections recognizing variations in the prevailing traffic conditions in non-congested and/or congested environment.

Keywords – Moving observer method; differential Global Positioning System; Laser detectors; measurement of vehicle speed; measurement of vehicle length; instantaneous speed distribution

1. Introduction

1.1 The traditional method

The ‘moving observer method’ originally proposed in [14] is used to estimate simultaneously the average flow and travel time of traffic from measurements carried out by a vehicle traveling with and against-the-stream for non-congested conditions. The strong point of the method appears to be its capability to estimate average traffic parameters along a highway section and for long time periods of measurements rather than obtaining measurements at a point. Nevertheless, these calculations are carried out without determining the instantaneous speed distribution, while they imply that four assumptions hold. First, all vehicle speeds remain constant along the section. Therefore the stream may be decomposed in a number of substreams; each consists of vehicles traveling at a common and constant speed [13]. Wright [15] pointed out that any variation in speed of a specific vehicle met by the observer does not really influence the ‘tally’ number. Any group of a passive and an active overtaking provides a null contribution to it, which is equivalent
with the fact that the observer would not meet such a vehicle if it traveled at its average speed. Second, all vehicles travel in uniform and constant spacing. Wardrop [13] argued that this assumption does not hold, and oppositely claimed that, since vehicles pass a given point in random, his traffic model ensures that the spacing between the successive vehicles forms a random series. However, Hall [5] discussing the derivation of the fundamental equation of traffic flow from the above model proved that all calculations are based on the implicit assumption of constant spacing. This assumption is necessary, since the period of observation of the individual substreams differs depending on their travel time. Therefore, when all substreams are summed up for the calculation of the total flow, stationarity of flow over time must be assumed in order to convert all subflows to the same time frame. Third, it is considered that the space speed distribution is stationary. It is expected that the road section is homogeneous and no change of traffic characteristics may occur during the various observer’s runs. Finally, the distribution is symmetrical and most likely normal. Thus, the observer’s speed is expected to approach the mean stream speed when the ‘tally’ number is zero. For non-congested traffic, the method doesn’t have the capacity to determine the fundamental parameters for short time periods (few minutes), failing in this way to fully replace point detector systems which, on the contrary, could do this. Even for long time periods (i.e. 1h-1.5h) it is found that 12-16 runs are necessary for estimating flow and mean speed. The above mentioned assumptions might hold true in case of light traffic and for long periods of time and long road sections, where the vehicles could develop freely or restore quickly their speeds and are not restricted by the motion of others. For congested roads several assumptions among those cited above are not valid. A collective speed is developed, which leads to absence of overtakings. In addition, stop-and-go traffic is observed, which may produce non-synchronized lane motions and resemble to overtakings to a moving observer but they are not relevant to flow determination. Therefore, the method in congested conditions isn’t reliable due to strong oscillating phenomena and could only be used by monitoring the moving observer’s speed. It might be assumed that it is representative for the speed profile of the traffic. Discussion on the use of probe vehicles for determining the travel time is actually based on this assumption of peak traffic. Recommendations to avoid measurements of this kind for minor arterials or for freeways during off-peak hours are based on the fact that in that case this assumption does not hold [11].

1.2 Motivation for revisiting the method

The traffic monitoring systems based on point detectors (loop detectors and/or cameras etc.) are very robust and extensively tested for their efficiency. Additionally, communication infrastructure already installed in several highway systems permits the transmitting of traffic data to Traffic Centers for deploying ITS strategies. Nevertheless, there are points to argue against their uniqueness. First, the cost of the infrastructure is high for installing detectors. For cutting down the cost, single presence detectors are in use, the accuracy of which, however, in determining the speeds is not very high. Second, point detectors capture localized data; Hall [5] proved that the harmonic mean speed thus calculated may considerably diverge from the mean space speed because it is unknown how many vehicles don’t cross the entire section. Therefore, the hypothesis that the harmonic mean speed approaches the true mean space speed holds under the implicit assumption that stationary conditions prevail between several parts of the stream (see chapter 3). In order to suppress the contribution of that assumption, a confined spacing of detectors is usually chosen; they are installed in distances between 500m and 1,5km. Point measurements succeed to determine the flows (the loading of the section) but not the space parameters (density and mean space speed) which qualify the traffic conditions in the highway.
section. A third problem discussed often in the last decade is that point measurements obscure the traffic dynamics [2], and a rather qualitative study of data from a series of point detectors could reveal the prevailing traffic characteristics of long highway sections [6]. Additionally, new needs and systems, i.e. ITS, require evaluation of the traffic parameters along road sections. Travel time (the inverse of the space speed) between interchanges is recognized as a principal parameter, which succeeds in describing freeway traffic [3]. It could be determined by successive point measurements [9,7] or vehicle techniques utilizing new technology [10]. The moving observer method, which may be capable to estimate the space speed distribution, is worthy to be studied. A number of probe vehicles are already rolling with the stream in the highways as service or police vehicles, which may be used for collecting data. The method proves its capacity, when fixed-detector and moving measurements are combined; then the exact speed distribution is determined and the exact mean space speed at the end of the measurement period could be calculated. For congested conditions the observer’s speed and its fluctuation may be considered as representative for the collective speed profile. It appears that GPS-based methods are appropriate for collecting such data [12]. Recognizing this need, the present paper attempts to enrich the traditional moving observer method in order to make it capable for accurate traffic parameter determinations for short time periods and non-congested conditions, and study its use to congested traffic conditions. In non-congested conditions, this is achieved by directly measuring the speed of all vehicles moving in one lane recorded by the observer moving in an adjacent lane with the aim to determine the prevailing speed distribution. Additionally, it makes possible to measure all vehicle lengths performing traffic synthesis determination. A DGPS-based subsystem and laser detectors constitute the measuring system that provides the required accuracy for determining vehicle speeds and lengths at an equal level of fixed in-road presence-type detectors. Furthermore, the possibility has been studied for the observer to follow the stream in an adjacent lane to its motion and recognize changes in congested non-synchronized lanes or when stop-and-go processes prevail, similarly to the way they are perceived by a driver moving in an adjacent lane.

2. The new moving observer method

2.1 The new method applied in non-congested traffic conditions

The moving observer method produces the estimates of interest without determination of the speed distribution. It is assumed that the mean speed could be approached by a self-action of the speed distribution, which is completed when the ‘tally’ number becomes zero. Oppositely, the method proposed in this paper carries out direct measurements of individual speeds with the aim to determine their distribution. In addition, it measures the length of each vehicle met by the observer, thus providing information about the traffic synthesis. The observer’s vehicle moves in one lane and measures the speed and the length of every vehicle moving in an adjacent lane. Exploiting the fact that all speeds prevailing in every lane differ considerably from those of its adjacent lanes, it is expected that the observer overtakes all vehicles of a slower lane and is overtaken by all vehicles of a faster lane. Due to technical as well as physical constraints the system carries out measurements only in the next-adjacent lane(s) to that of the moving observer. The plural refers to measurements conducted simultaneously to the adjacent right and left lanes of a multilane highway while the observer moves in-a-between lane.

Further on, the recorded vehicles are assigned in speed classes. The numbers thus calculated are further divided by appropriate time frames providing the substream flows. Accepting the stationarity of flow over time for every speed class, the number of vehicles that remain in the
section at the end of the period of observer’s motion could be determined. These values represent the frequencies of the space speed distribution at that point in time.

2.2 Basic assumptions

Detecting any vehicle at a random position, the method could not ensure that the measured speed is its average speed. Therefore it accepts the first assumption of constant speeds as prerequisite for determining the speed distribution shape. The second assumption of constant spacing is also adopted, since stationarity of flow over various time periods is required. The two first assumptions further lead to the acceptance of the stability of the fundamental traffic law, which relates the fundamental parameters. The stability of the speed distribution is an assumption that could be made without loss of generality because the total time of measurement is reduced to few minutes. It is widely accepted that speed is normally distributed. However, several investigators found speed distribution to be quite skewed when modelled by the normal distribution [e.g. 4]. Furthermore, the time and space speed distributions could be both assumed as normal. Breiman et al. [1] state that their difference is not important; thus, within the limits of accuracy of all other assumptions this does not introduce a considerable error. These observations are necessary for supporting the traditional method which relates data collected during long period of measurements. Contrary, for collecting data for short periods it is not necessary to presuppose the distribution as being symmetrical and normal.

2.3 Time period of measurement of each speed class

In Figures 1 and 2 the period of measurement of a ‘faster’ substream flow ($t_{mean,f,i}$) and of a ‘slower’ substream flow ($t_{mean,s,i}$) are demonstrated respectively, when the observer moves with-the-stream at a mean speed $\bar{u}_w$ and for a period of $t_w$. If $t_i$ and $u_i$ denote the period of motion and speed of the observed substream vehicles, then:

$$t_{meas,f,i} = (t_w - t_i) = t_w \frac{u_i - \bar{u}_w}{u_i} = t_i \frac{u_i - \bar{u}_w}{\bar{u}_w}$$ \hspace{1cm} (1)

$$t_{meas,s,i} = (t_i - t_w) = t_w \frac{\bar{u}_w - u_i}{u_i} = t_i \frac{\bar{u}_w - u_i}{\bar{u}_w}$$ \hspace{1cm} (2)

![Fig. 1 – Calculation of faster substream flow using the method with-the-stream and against-the-stream](image-url)
2.4 Calculation of substream flow

The substream flow could be calculated by the following equation 4, since the stationarity of flow over time and speed along the section holds. \( (m_i) \) denotes the number of vehicles counted by the moving method and \( (n_i) \) the number of vehicles that remains in the section at the end of the measurement. The latter equals the number of vehicles that cross the point during the time of motion of every speed class along the section. For the substream flow \( (q_i) \) holds:

\[
q_i = n_i = \frac{m_i}{t_i} \quad \Rightarrow \quad n_i = m_i \frac{t_i}{t_{\text{meas},i}}
\]

Consequently, the number of vehicles found in the section at the end of the measurement period for the ‘faster’ substreams \( (n_{f,i}) \) and the ‘slower’ substream \( (n_{s,i}) \) are calculated as follows, using the observer moving measurements with-the-stream \( (m_{f,i}) \) and \( (m_{s,i}) \) respectively:

\[
n_{f,i} = m_{f,i} \frac{u_w}{u_i - u_w}
\]

\[
n_{s,i} = m_{s,i} \frac{u_w}{u_w - u_i}
\]

By the above equations the faster and the slower substream flow could be calculated for with-the-stream motion, \( q_{f,i} \), and \( q_{s,i} \), respectively:

\[
q_{f,i} = \frac{m_{f,i} u_i}{u_i - u_w} \frac{1}{t_w}
\]

\[
q_{s,i} = \frac{m_{s,i} u_i}{u_w - u_i} \frac{1}{t_w}
\]

Fig. 2 – Calculation of slower substream flow using the method with-the-stream and against-the-stream

\( (t_{\text{meas},i} \): period of measurement for motion with-the-stream)
In all cases the fluctuation of the observer speed isn’t a source of error. As in the traditional method, any couple of vehicles consisting of an overpassed and an overpassing vehicle must be omitted from the calculation. Therefore, the data collected by the new method for any speed class is a ‘tally’ number as well. For this reason the average observer speed is considered in the above calculations.

2.5 Qualification of the new method applied in non-congested traffic conditions

The moving observer meets a small percentage of vehicles moving at speeds around his/her value and a large percentage of vehicles moving at very different speeds. These percentages vary according to the speed classes. Therefore, the method collects more reliable data about the tails of the speed distribution rather than from its main part. Equations 5 and 6 then make a transformation to correct the speed distribution and to restore it to its normal shape. However, in realistic terms (observer’s speeds: 60km/h-100km/h and vehicle speeds: 60km/h-140km/h) these transformations require an average factor of 3-5 for predict the number (N) of vehicles moving in the freeway from those detected (M) by the observer. Therefore, the method seems to perform a low prediction power, since from 20%-35% of the stream attempts to predict the total number of vehicles of the stream. One way to improve its prediction power is to increase the distance of measurement that results to an increase of the number of vehicles of the detected samples. This leads to calculate the mean speed in a long freeway section (i.e. 1.5km), which supposedly is an advantage of the method [14], instead of calculating the fundamental variables in short pieces of freeway that the fixed-detector system does. This drawback of the method may be considered a-priori as a contra-motivating characteristic for further exploring it. However, a second observation discussed at the next section raises the interest about its development. In all techniques developed below for implementing the method the observer meets distinct group of vehicles, while the fixed-detector counts a large number of vehicles but without distinction at which group they belong. Therefore, the combination of a fixed-detector and a moving observer’s measurements could assign every vehicle to its group. Then the exact speed distribution that prevails at the end of the measurement period could be determined.

2.6 The new method applied in congested traffic conditions

The method has the capability to measure the speed of the vehicles met by the observer even in congested conditions. In a macroscopic approach the entire stream runs at a collective speed, however in microscopic view variations of speeds between vehicles are realized especially when non-synchronized lanes are observed and stop-and-go traffic prevails. The measurement technique is based on positioning and timing determinations of the various events (activations and deactivations of the detectors; see for definitions in chapter 4), as well as on the observer’s speed determination. The GPS measurements are provided on a specific rate (in the experimental work of this paper every 1sec), data that could be used to determine an average speed of any vehicle met during the various events. Eventually, the observer moving in one lane could follow the traffic in the adjacent(s) lane(s), similarly to the way they are perceived by a driver moving in congested conditions.

Contrary, the method cannot ensure the measurement of flow. Macroscopically, since a collective speed prevails the observer doesn’t meet any vehicle. Microscopically, the multiple overtakings (the same vehicles move back and forth with respect to the observer) do not allow the calculation of flow.
3. Description of four techniques for implementing the method

The vehicles that move in a time-space domain (i.e., in a section between two point detectors during a period of few minutes) belong to four groups:

EX: the vehicles that are in the section and exit during the period of measurement.
P: the vehicles that enter and exit the section during the period of measurement.
R: the vehicles that remain in the section during the entire period of measurement.
EN: the vehicles that enter and remain in the section during the period of measurement.

N = EN + R: the vehicles that are in the section at the end of the measurement period.

At a point detector only the vehicles P and EN are counted. Often, for non-congested traffic the R vehicles don’t exist. For example, for a section of 1.7 km and period of measurement of 2 min, a vehicle that covers the distance during the entire period must move at 50 km/h (=1.7 km/2 min). Since all vehicles are expected to move at a speed greater than 70 km/h, none could be of R type; contrary R vehicles move in the section when the period of measurement is 1 min (100 km/h = 1.7 km/1 min). In general, the harmonic mean of all measured vehicle speeds, as representative of space speed, suffers from four problems. First, it is considered that all speeds remain constant along the section. Second, only P and EN vehicles are counted, without distinguishing which vehicle belongs to each group. Third, some vehicles don’t cover the entire section distance, which is equivalent to consider that the traffic conditions of EX and EN are identical. This is what Hall [5] proved by stating that the harmonic mean speed thus calculated suffers because it is unknown how many vehicles don’t cross the entire section, i.e., the size of this latter group. Nam and Drew [7] proposed the P group to be considered as representative for the entire stream. In order to reinforce the argument they proposed the extension of the time of measurement up to 5 min, in order to relatively suppress the existence of the EN vehicle group and eliminate the R vehicle group. Fourth, it is known that when measuring at a point the fast vehicles are over-presented and the slow vehicles under-presented compared with those in the section at any point in time. Therefore, the consideration that the harmonic mean speed of P + EN vehicles equals to the arithmetic mean speed of EN vehicles implies an error.

The new observer method could be implemented following four different techniques.

**Technique 1: Observer traveling within the stream**

The technique could be used in the conventional way. However, it is not based on the ‘tally’ number measurement. The observer counts the ‘faster’ and ‘slower’ vehicles and measures their individual speeds. He/she gathers significant information about the tails of the distribution, but very limited about the speed classes around his/her speed. Integrating all available information, the total flow results from the sum:

\[
Q = \sum_i q_i = \sum_{s,i} m_{s,i} \frac{u_i}{u_w - u_i} t_w + q_w + \sum_{f,i} m_{f,i} \frac{u_i}{u_i - u_w} t_w
\]  

(8)

\(q_w\): the flow of the speed class whose value equals to the observer’s speed.

By one run the \((q_w)\) value remains unknown. Its calculation may be performed collecting data by an additional run carried out at a different speed, if it is possible.
The mean space speed is given by the arithmetic mean of the speed values of all vehicles found in the section at the end of the period of measurement:

$$
U_s = \frac{\sum_i n_i u_i}{\sum_i n_i} = \frac{\sum_{f,i} m_{f,i} \frac{u_w}{u_i - u_w} u_i + q_w t_w u_w + \sum_{s,i} m_{s,i} \frac{u_w}{u_w - u_i} u_i + q_w t_w u_w + \sum_{s,i} m_{s,i} \frac{u_w}{u_w - u_j} u_j}{\sum_{f,i} m_{f,i} \frac{u_w}{u_i - u_w} u_i + q_w t_w u_w + \sum_{s,i} m_{s,i} \frac{u_w}{u_w - u_i} u_i + q_w t_w u_w + \sum_{s,i} m_{s,i} \frac{u_w}{u_w - u_j} u_j}
$$

(9)

Regarding the tail consisting of the faster substreams, the observer could meet all P and all R vehicles within the moving observer time of measurement along the section. However, the first technique suffers from all the problems that the traditional method does; mainly, it lacks information about the speed classes around the observer’s speed. When the section is short, it leads to unbalanced presentation of the two tails of the speed distribution. When the section is extended, then the P vehicles tend to be equal to the R vehicles, decreasing up to zero the tally number. Then the speed distribution is expected to be symmetrical and the calculated mean space speed to be reliable. The combination of measurements from a fixed-detector situated at the beginning of the section and moving observer provides the exact number of EN vehicles and their speeds. Then the addition of R and EN vehicles determines the exact speed distribution at the end of the measurement period.

However, this technique determines a general all-lane distribution (for freeway two or three lanes). Since nowadays it is expected an independent determination of the fundamental variables to be carried out for every lane, two new techniques were studied by drawing out the observer from the observing stream; then the observer moves slower or faster than all the lane speed classes.

**Technique 2: Observer traveling with-the-stream at a slower speed**

In practice and due to the technical constraints, the observer moves in one lane and collects data of the next-adjacent lane (measurements concerning the slow lane could be carried out while the observer moves along the emergency lane if this is possible). According to the previous discussion he/she could then meet realistically all speed classes of one lane. Therefore, a separate vehicle is required for every individual lane in order to determine the total flow of the section.

The data thus collected don’t constitute a sample. The observer meets all P vehicles within the moving observer period of measurement along the section. R vehicles don’t exist since all vehicles are faster than the observer. Further on, by transforming the raw data ($\Sigma m_i$) to EN vehicles, ($\Sigma n_i$) in number, using equation 5 the mean space speed could be calculated. This transformation assumes that the substream conditions of P and EN vehicles are identical. The two variables of traffic stream for the period of measurement are calculated for the ($j$) lane as follows:

$$
Q_j = \sum_i m_{i,j} \frac{u_{i,j}}{u_{i,j} - u_{w,j}} t_{w,j}
$$

(10)
The combination of moving observer with point detector measurements can definitely determine which vehicles are of EN type and P type. Knowing the real number of EN vehicles the exact space speed distribution is calculated at the end of the measurement period.

**Technique 3: Observer traveling within the stream at a faster speed**

This technique could be used to collect information for a lane right-adjacent to that of the observer. The part of the distribution visited this time is the left, that of slow vehicles. The data thus collected don’t constitute a sample. The observer meets all R vehicles within the moving observer period of measurement along the section.

The 3d technique most probably couldn’t collect measurements for the median lane because in general the inner shoulder is not wide enough for vehicle motion. Nevertheless, the merit of the technique is that no double counting of overtaken vehicles occurs. The two variables of traffic stream for the period of measurement are calculated for the \( (j) \) lane as follows:

\[
\overline{U}_{s,j} = \frac{\sum_{i} m_{i,j} u_{i,j}}{\sum_{i} m_{i,j}} \frac{u_{i,j}}{u_{w,j} - u_{i,j}}
\]

(11)

\[
\sum_{i} m_{i,j} u_{i,j}
\]

\[
\sum_{i} m_{i,j} u_{i,j}
\]

\[
\sum_{i} m_{i,j} u_{i,j}
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\[
\sum_{i} m_{i,j} u_{i,j}
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\sum_{i} m_{i,j} u_{i,j}
\]

\[
\sum_{i} m_{i,j} u_{i,j}
\]

Combining the moving observer and point detector measurements all R+EN vehicles and their relative speeds are known. Then the exact speed distribution at the end of the measurement period is determined.

4. Measurement technique and technology

4.1 Measurement requirement

The speed and length determinations by a fixed detector rely on the occupancy time and the length of the detector. In order to establish a moving method that could measure speed with high accuracy, a similar concept has been realized. The time in which a vehicle overpasses another vehicle, obviously depending on their speed difference, lasts from hundreds msec up to few seconds. For instance, if the speed difference is about 10km/h or 100km/h and both vehicles have a length of about 4m, the overtaking is realized in 2.88sec and 0.288msecs, respectively. For longer vehicle(s) the time period increases accordingly. The above range indicates that a system
that could carry out measurements of time of this magnitude must perform faster by one order, i.e. few tenths of msec.

4.2 System configuration

The new moving observer technique is based on its ability to measure the speed and the length of each vehicle moving adjacently to the observer’s vehicle. Two detectors are placed at a specified distance at the side of the observer’s vehicle. They are activated (ON) and deactivated (OFF) when a vehicle starts and ends moving next to the observer. While any detector is ON the time is counted. In parallel, the moving observer is equipped with a DGPS-based system, which could determine its position and speed at UTC (Coordinated Universal Time). The position and the timing of four discrete events indicating the activation and deactivation of the two detectors are determined. The GPS time updated in short time slices (i.e. every second) could synchronize all other components of the system through a PLC (programmable logic controller). The configuration of such a generic system is described in Figure 3. There are four subsystems, namely, the PLC, the GPS, the detection part and a portable computer. The first subsystem consists of a PLC and is responsible for three tasks, namely the synchronization task, the time counting while the detectors are ON, and the data transmission to a computer for being stored. The first two tasks could be carried out in few msec, while the latter needs 50-70 msec.

The second subsystem consists of a GPS, using the RTK/OTF technique with differential corrections. It determines the universal time with an accuracy of nsec, provides positioning every 1 sec with an accuracy of 1cm, and assures accuracy of the speed of the moving observer of ±2‰ per sec (±0.02m/10m per sec). The GPS subsystem transmits every second the universal time to the PLC. The synchronization is realized through a pulse. The PLC counts the time difference between the exact second and the activation of any detector. Additionally, it counts the time period that the detector is ON. Finally, the message it transmits to the computer contains the GPS information plus the time periods counted and the universal time of the activation and deactivation of the detector. The communication capacity permits these messages to be transmitted and stored at the computer (the fourth subsystem) in tenths of msecs. This last constraint does not affect the error propagation for the determination of the traffic parameters, but specifies the necessary discrete time gap for not confusing the successive measurements (see below paragraph 6). Regarding the third subsystem, industrial laser detectors reliably working outdoors (day and night) are used, covering a range of several meters (2.5-6m) without reflector (diffusive type). The maximum range up to 6m could be regulated according to the need of the experiment (i.e. shorter for a narrow lane). They could be activated in 1msec. Finally, the fourth subsystem acts as a storage device.

Fig. 3 – Physical components of the measuring system
Thus, it seems that a measurement could be conducted in very short time without being confused with the preceding one. In parallel, the speed and the positioning of the moving vehicle are available in discrete time and in asynchronous process, since they are updated every second.

4.3 Determination of vehicle speed and length

Such a moving system may be considered as a moving double presence-detector. The new technique could calculate the speed and the length of any moving vehicle that overpasses the observing vehicle or is overtaken by it. Four cases are distinguished while the observer moves with-the-stream: faster and longer vehicles (case 1); faster and shorter vehicles (case 2); slower and longer vehicles (case 3); slower and shorter vehicles (case 4). The term ‘shorter’ or ‘longer’ characterizes the vehicle whose length is respectively shorter or longer than the distance between the two detectors (trap base: \(\mu_o\)). In Table 1 the sequence of events (activation and deactivation of the detectors) are described, where it is apparent that every case is governed by a specific and fully recognizable pattern. In Table 2 the speed and length calculations are given. For explaining the derivation of the equations case 1 and 4 are described graphically (Figures 4 and 5).

The new technique has the capability to measure the length and the speed of the vehicle, which may change during the passage. In order for the change to be detected, the process must last for an adequate time period (i.e. few minutes), and obviously concerns a vehicle moving at a slightly different speed than the observer’s speed. Then the vehicle length could be calculated assessing which is the intermediate value (\(u_2\)) of the acceleration or deceleration movement. The difference (\(\Delta t_{23}\)) provides information about the type of motion, and (\(\Delta t_{23}\)) about the length difference of the vehicle to the trap-base. In all cases the vehicle length equals to the trap-base when (\(\Delta t_{23}\)) equals zero. The system used for the experimentation work of the present study counts the time period for which a detector is active (the front or rear detector is active respectively: \(\Delta t_{\text{Front,ON,OFF}}\), \(\Delta t_{\text{Rear,ON,OFF}}\)). In this case, rewriting the equations of Table 2 a common general set of equations is formed for all cases (\(u_o\) is the instantaneous observer’s speed).

<table>
<thead>
<tr>
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<th>1</th>
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<th>3</th>
<th>4</th>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>UTC_\text{front,OFF}</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Faster/longer</td>
<td>D2 (rear)</td>
<td>ON</td>
<td>OFF</td>
<td>OFF</td>
<td>OFF</td>
</tr>
<tr>
<td></td>
<td>D1 (front)</td>
<td>OFF</td>
<td>OFF</td>
<td>ON</td>
<td>OFF</td>
</tr>
<tr>
<td>Time evolution</td>
<td>UTC_\text{rear,ON}</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>UTC_\text{front,ON}</td>
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</tr>
<tr>
<td></td>
<td>UTC_\text{rear,OFF}</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>UTC_\text{front,OFF}</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Faster/shorter</td>
<td>D2 (rear)</td>
<td>OFF</td>
<td>ON</td>
<td>ON</td>
<td>OFF</td>
</tr>
<tr>
<td></td>
<td>D1 (front)</td>
<td>ON</td>
<td>ON</td>
<td>OFF</td>
<td>OFF</td>
</tr>
<tr>
<td>Time evolution</td>
<td>UTC_\text{rear,ON}</td>
<td></td>
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<tr>
<td></td>
<td>UTC_\text{front,ON}</td>
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<tr>
<td></td>
<td>UTC_\text{rear,OFF}</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>UTC_\text{front,OFF}</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slower/longer</td>
<td>D2 (rear)</td>
<td>OFF</td>
<td>ON</td>
<td>ON</td>
<td>OFF</td>
</tr>
<tr>
<td></td>
<td>D1 (front)</td>
<td>OFF</td>
<td>OFF</td>
<td>ON</td>
<td>OFF</td>
</tr>
<tr>
<td>Time evolution</td>
<td>UTC_\text{rear,ON}</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>UTC_\text{front,ON}</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>UTC_\text{rear,OFF}</td>
<td></td>
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<tr>
<td></td>
<td>UTC_\text{front,OFF}</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Slower/shorter</td>
<td>D2 (rear)</td>
<td>OFF</td>
<td>ON</td>
<td>ON</td>
<td>OFF</td>
</tr>
<tr>
<td></td>
<td>D1 (front)</td>
<td>OFF</td>
<td>OFF</td>
<td>ON</td>
<td>OFF</td>
</tr>
<tr>
<td>Time evolution</td>
<td>UTC_\text{rear,ON}</td>
<td></td>
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<td>UTC_\text{front,ON}</td>
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<tr>
<td></td>
<td>UTC_\text{rear,OFF}</td>
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<td></td>
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<tr>
<td></td>
<td>UTC_\text{front,OFF}</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tab. 1 – Recognizable patterns of time sequence of events
Fig. 4 – Evolution of events during the meeting of a faster and longer vehicle with the observer traveling with-the-stream (case 1); $P_i$ are the positionings of the events and UTC$_i$ the timings of the events.

For faster vehicles:

\[
\mu_i = \mu_0 + \frac{\mu_0}{\text{UTC}_\text{front,ON} - \text{UTC}_\text{rear,ON}} = \mu_0 + \frac{\mu_0}{\text{UTC}_\text{front,OFF} - \text{UTC}_\text{rear,OFF}} \\
\mu_i = \mu_0 \frac{\Delta t_{\text{rear,ON} - \text{OFF}}}{\Delta t_{\text{front,ON} - \text{rear,ON}}} = \mu_0 \frac{\Delta t_{\text{front,ON} - \text{OFF}}}{\Delta t_{\text{rear,OFF} - \text{rear,OFF}}} 
\]

Fig. 5 – Evolution of events during the meeting of a slower and shorter vehicle with the observer traveling with-the-stream (case 4); $P_i$ are the positionings of the events and UTC$_i$ the timings.

For slower vehicles:

\[
\mu_i = \mu_0 - \frac{\mu_0}{\text{UTC}_\text{rear,ON} - \text{UTC}_\text{front,ON}} = \mu_0 - \frac{\mu_0}{\text{UTC}_\text{rear,OFF} - \text{UTC}_\text{front,OFF}} \\
\mu_i = \mu_0 \frac{\Delta t_{\text{front,ON} - \text{OFF}}}{\Delta t_{\text{rear,ON} - \text{front,ON}}} = \mu_0 \frac{\Delta t_{\text{rear,ON} - \text{OFF}}}{\Delta t_{\text{rear,OFF} - \text{front,OFF}}} 
\]

Recorded vehicle speed could be calculated even if the observer’s speed fluctuates. This is possible because GPS measurements could be provided for every time period ($\Delta t_{12}$, $\Delta t_{23}$, $\Delta t_{34}$). Therefore the speeds $u_1$, $u_2$, $u_3$ of Table 3 could be determined. Contrary, any fluctuation of the observed vehicle speed influences the determination of its length.

5. Accuracy of variables

The capability of the method to measure the specific determinants arises because the GPS technology provides accurate measurements of time and speed of the observer. Studying the relationships provided in Table 2 it is understood that calculation of speed from a short space unit ($\mu_i$) could be achieved only if a very accurate determination of time is provided. This is feasible using this technique since the time accuracy is of the msec level.
Tab. 2 – Speed and length determination of the observed vehicle

<table>
<thead>
<tr>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( u_1 = \frac{\Delta t}{\Delta t_1} )</td>
<td>( u_1 = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1 + \Delta t_2 + \Delta t_3} )</td>
<td>( u_1 = \frac{\Delta t}{\Delta t_1} )</td>
<td>( u_1 = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1 + \Delta t_2 + \Delta t_3} )</td>
</tr>
<tr>
<td>( u_1 = \frac{\rho u_1}{\Delta t_1} )</td>
<td>( u_1 = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1 + \Delta t_2 + \Delta t_3} )</td>
<td>( u_1 = \frac{\rho u_1}{\Delta t_1} )</td>
<td>( u_1 = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1 + \Delta t_2 + \Delta t_3} )</td>
</tr>
<tr>
<td>( u_1 = u_0 + \rho u_1 )</td>
<td>( u_1 = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1 + \Delta t_2 + \Delta t_3} )</td>
<td>( u_1 = u_0 + \rho u_1 )</td>
<td>( u_1 = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1 + \Delta t_2 + \Delta t_3} )</td>
</tr>
</tbody>
</table>

If \( \Delta t_1 = \Delta t_3 \), then \( u_1 = u_2 = u_3 \)

\[ \mu = \frac{\Delta t_1 + \Delta t_2}{\Delta t_1} \]

The error analysis of both equations 14 and 15 leads to identical equations, namely:

\[
\sigma_u^2 = \left( \frac{\partial u}{\partial u_0} \right)^2 \sigma_{u_0}^2 \left( \frac{\partial u}{\partial \mu_0} \right)^2 \sigma_{\mu_0}^2 + \left( \frac{\partial u}{\partial \Delta t} \right)^2 \sigma_{\Delta t}^2
\]

\[
\sigma_\mu^2 = \sigma_{\mu_0}^2 + \left( \frac{\Delta \mu^2}{\mu_0^2} \right) \sigma_{\mu_0}^2
\]

Carrying out the error analysis, it appears that the expected accuracy of the speed determination is influenced basically by the last component of equation 16. The observer’s speed and the trap base are determined very accurately, letting only time to contribute to the speed accuracy. For instance, in the experimental work of this paper the speed accuracy is within ±1.5km/h for a speed difference of 100km/h between the observer and the observed vehicle (Table 3). Moreover, the length accuracy is significantly high for all cases, being better than ±0.5m. The last component of equation 17 mainly contributes to the length error, which is subject to the time determination of independent events. Although theoretically the above values are very satisfactory, in practice the laser performance is significantly influenced by several factors not easily and generally quantified. The varying environment in terms of sun lighting, humidity and gas existence affects the lasers, and the same is true for the types of reflecting surfaces (vehicle color, condition of the surface, height and geometric variation of the surface, especially of buses).
These factors affect laser activation and deactivation, and may as well lead to numerous short-lived measurements. Such sources of error encountered in the field produce noisy measurements and errors in time estimations. For this reason, in the experiments of this work a great deal of effort was spent to filter and distinguish the real measurements. A video camera was used in order to group short measurements and distinguish processes and vehicles.

Additionally, the detection method needs some care, since the laser beam displacement may contribute to a serious error. In the experiments of this work, its value has been determined by accurate experimental calibration at the field, using a mechanism for rendering each beam perpendicular to the lateral surface of the observer vehicle, and parallel to each other.

### 6. Rules for distinguishing the vehicles

#### 6.1 Rules for distinguishing the vehicles in unconstrained flow conditions

The time sequence of the activation and deactivation of the two detectors are indicated in Table 1. As mentioned above, the system used in the experiments of this work simplifies the sequence, since the timings of activation and deactivation of each detector in addition to the period of activation are packed together and stored simultaneously. Then it is easier to recognize which set of the 1st detector must be related to the corresponding set of the 2nd detector, in order to reproduce the process of the passage of every vehicle. Otherwise it would be harder and in some cases uncertain to recognize which ‘ON’ and ‘OFF’ of each detector refers to the same vehicle, and thus which process is described. When putting together detector data for a series of processes a problem of how to distinguish them arises. In Table 1 the sequence of detector data are given for vehicles of three different length classes in relation to the data of different speed classes. It appears that for all these combinations vehicles follow processes characterized by specific sequences of phases that have recognizable patterns. However, in practice questions may appear whether there is an error in considering the first detector activation in a series of processes. It is expected that this error is generally avoided in most flow conditions, since the prevailing headways are significantly greater than the detector activation time. Thus, there are clear and distinguishable time gaps between vehicle movements leading to distinction between vehicle processes and phases. To reinforce this statement, it must be noted that the minimum value of the expected headway between two passenger vehicles is around 0.4 sec. This value becomes 0.5 sec when a heavy good vehicle is involved, and up to 0.7 sec when both vehicles are of this kind [8].

In general, the distinction of one vehicle pattern from the others is embedded in two rules. The first rule refers to the process of activation/deactivation, while the observer moves with-the-stream, which evolves in two ways depending on the relative speed of the two vehicles. When the observer overpasses a vehicle then the front detector is activated first and next the rear one. The opposite is true when the observer is overtaken. A second rule refers to the timing of deactivation of the first detector. When the observing vehicle is longer than its adjacent one then there is an

<table>
<thead>
<tr>
<th>( \mu_0 ) (m)</th>
<th>180</th>
<th>145</th>
<th>110</th>
<th>90</th>
<th>70</th>
<th>55</th>
<th>35</th>
<th>20</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Delta u ) (km/h)</td>
<td>3.5</td>
<td>5.2</td>
<td>3.3</td>
<td>1.9</td>
<td>1.3</td>
<td>0.8</td>
<td>0.5</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>4.5</td>
<td>2.9</td>
<td>1.6</td>
<td>1.1</td>
<td>0.7</td>
<td>0.4</td>
<td>0.2</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>6</td>
<td>5.0</td>
<td>1.9</td>
<td>1.1</td>
<td>0.8</td>
<td>0.5</td>
<td>0.3</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>10</td>
<td>1.8</td>
<td>1.2</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
<td>0.2</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
</tbody>
</table>
intermediate time period when both detectors are ‘OFF’. Oppositely, when the observing vehicle is shorter then there is a time period both detectors are ‘ON’. If the first phase is completed when the first detector is deactivated and the second phase lasts as long as the second detector is active, then the patterns indicated in table 1 are unique and fully recognizable. When the observer moves in the opposite direction of the stream the process is similar to the case of the slower vehicle. As mentioned elsewhere, the difference results from very short trap times. Theoretically, before the second detector is deactivated a following vehicle (depending on speed difference of the observed and observing vehicle) may complete its 1st phase (i.e. activation and deactivation of the first detector). Because the selected system clusters the ‘ON’ and ‘OFF’ signals and then transmits them as a package, such an intercalary phase of the other vehicle may appear only when the intervehicle headway is very short. In non-congested conditions the problem arises marginally for extreme unfavorable cases; for instance considering the minimum expected intervehicle headway to be around 2.5m at low speed, i.e. 20-30km/h [8], a reasonable minimum vehicle length of about 3.5m, and the trap length greater than 6m. Oppositely, under congested conditions the observed vehicle may travel at very low speed and short headway, thus even a shorter trap length of about 4-5m permits the completeness of the 1st phase of the following vehicle before that of the 2nd phase of the preceding vehicle. Therefore, a methodology for distinguishing the vehicles could be based on the following steps. First, after determining a significantly large headway a bunch of vehicles is recognized. Second, working within each bunch, directions of movement and the level of the relative speed (faster or slower) are determined. Additionally, intercalated phases could be distinguished. This step concludes in clustering the detector measurements that refer to the same vehicle. Third, the sign of the intermediate time period between the deactivation of the first detector and the activation of the second detector ($\Delta t_{23}$) indicates whether the observed vehicle is shorter or longer than the trap-base. Four, the difference of the other two time periods ($\Delta t_{12}$, $\Delta t_{34}$) in relation to the observer’s speed assesses whether the observed vehicle conducts a deceleration/acceleration maneuver or travels at a constant speed. Finally, the calculation of the speed and the length could be carried out.

6.2 Rules for distinguishing the vehicles in congested flow conditions

Obviously, special cases occur when the observed vehicle would finally not realize the entire process. These cases refer to a series of deceleration/acceleration maneuvers, common in stop-and-go conditions. It could also occur when the observing vehicle changes its speed at such a level that during the detection a faster vehicle becomes a slower vehicle or the inverse. Various patterns of detector data could be formed as shown in table 4, namely incomplete execution of the first phase, incomplete execution of both first and second phases, complete execution of 1st phase and incomplete execution of the 2nd phase, and finally complete execution of the entire process but initially acceleration (deceleration) and then the opposite, most probably realizing an unreasonable short time headway.

Examining the possible patterns, it is understood that in several cases a process of another vehicle may be intercalated within the pattern. These cases illustrated in table 3 are the two last processes when the observing vehicle is shorter; the three last processes when it is of equal length, and almost all when it is larger. This implies that the favorable indication that larger observer’s vehicle leads to more accurate determination of the parameters under unconstrained flow conditions is counterbalanced by uncertainty in distinguishing the vehicles and the processes under congested conditions. Obviously, different trap bases may be used for the various conditions. It is possible to have 3 detectors constituting two trap-bases (a short and a long one)...
and select those data that appear more reasonable in every case. However, this introduces further intelligence to the measuring equipment whose complexity could not be estimated at the present time (no experimentation of this type has been conducted). Although several difficulties appear in distinguishing the vehicles and the processes when their phases are mixed, certain generic rules could still be developed. Basically, conclusions may be supported by the specific vehicle measurement and the behavior of the surrounding vehicles.

### Tab. 4 – Patterns of incomplete processes

<table>
<thead>
<tr>
<th>Speed of vehicles (observed &amp; observing)</th>
<th>( v_i &gt; v_o )</th>
<th>( v_i = v_o )</th>
<th>( v_i &lt; v_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>( D_{rear} )</td>
<td>( D_{front} )</td>
<td>( D_{rear} )</td>
</tr>
<tr>
<td>Length of vehicles (observed &amp; observing)</td>
<td>Process</td>
<td>( \mu &gt; \mu_o )</td>
<td>( \mu = \mu_o )</td>
</tr>
<tr>
<td>Incomplete execution of 1st phase</td>
<td>( ON )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 1st and 2nd phases</td>
<td>( ON )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two 1st phases</td>
<td>( ON )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 2nd phase</td>
<td>( ON )</td>
<td>( ON )</td>
<td>( ON )</td>
</tr>
<tr>
<td>Execution of two complete processes by the same vehicle</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 1st phase</td>
<td>( OFF )</td>
<td>( ON )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two 1st phases</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 2nd phase</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two complete processes by the same vehicle</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 1st phase</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
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<tr>
<td>Execution of two 1st phases</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 2nd phase</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two complete processes by the same vehicle</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>( \mu = \mu_o )</td>
<td>( ON )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two 1st phases</td>
<td>( ON )</td>
<td>( ON )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 2nd phase</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two complete processes by the same vehicle</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>( \mu &lt; \mu_o )</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two 1st phases</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Incomplete execution of 2nd phase</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
<tr>
<td>Execution of two complete processes by the same vehicle</td>
<td>( OFF )</td>
<td>( OFF )</td>
<td>( OFF )</td>
</tr>
</tbody>
</table>
For instance, the non-execution of all phases, which indicates an acceleration/deceleration maneuver of the specific vehicle, is reasonably expected to be followed by a similar change in the relative speed of the next vehicles or the preceding vehicle. Furthermore, although the adjacent lanes may behave differently (no synchronized lanes), the observing vehicle speed is expected in the long run to fluctuate also in a similar way. Another indication is an unexpected difference between the time periods $\Delta t_{13}$, $\Delta t_{34}$. This could be compared with the possible deceleration or acceleration capabilities of the vehicles. Obviously, when the time periods are unrealistic then a safe conclusion could be that the observing and the observed vehicles travel at a common speed. Using these rules it is possible to study even a stop-and-go flow. Incomplete processes may occur and, thus, assumptions must be applied in order to distinguish the vehicles and the processes.

6.3 Selection of the trap-base

The trap-base could be selected by examining its contribution to the determination error of the basic variable (Table 3). Although its precision could be stable at the level of 1mm, the contributing error depends on its magnitude. Obviously, a larger value reduces considerably this contribution leading to a conclusion that for unconstrained flow conditions a large trap-base is favorable. Establishing a maximum acceptable error for speed of 2.5km/h, it seems that even a value of 3.5m is sufficient for speed differences up to 110km/h. Thus, a trap base of 4m, which is achievable using a passenger vehicle as observing vehicle, seems to be adequate for measuring speeds on a highway. Contrary, the trap-base contribution to the determination of vehicle length is not similarly important, mainly because the accuracy of this parameter is not so important. Thus, any length larger than 2-3m is sufficient for distinguishing vehicle classes even in contra-flow use of the technique. It is mentioned above that the trap-base length influences inversely the capability of the method when phases of different vehicles must not be mixed. Shorter trap-base length distinguishes with higher certainty the vehicle processes. However, under unconstrained flow conditions the selection of longer trap-bases does not produce serious problems. It becomes important when the technique is used under congested traffic conditions.

7. Applications carried out with the new method

7.1 Experimental work for traffic parameter determinations in non-congested conditions

The purpose of the experimental work presented herein is to demonstrate the capability of the method to collect data that are sufficient to determine the fundamental parameters of flow and speed using the 2nd technique. Several experiments have been carried out with the observer moving in the middle or the right lane of a 6-lane highway in order to observe the traffic in the left and middle lane respectively. The observer’s speed has been retained at a speed of 10km/h-15km/h lower than that of the middle lane under observation and 20km/h-25km/h lower than that of the left lane, however it varied within ±5km/h around its mean following the lane traffic.

Synchronizing the GPS time with the detectors’ time, the instantaneous observer’s speed was known allowing the determinations of the speed and length of the observing vehicle to be carried out with the accuracy expected by the theory developed above. Because the traffic at the observing lane was videoscoped, all vehicles have been recognized and their true lengths have been taken from manufacturers’ lists. Then the true lengths have been compared with those determined by the present method. In some experiments where discrepancies were revealed in random cases, the true length has been considered for the calculation of the specific vehicle speed. However, there were experiments where a systematic error appeared (more than one third of the
determined vehicles lengths deviated by more than 50cm). These experiments were excluded from further consideration. The most probable reason for these failures was the unfavorable environmental conditions that did not allow the detectors to perform properly.

Data have been collected in a 6-lane divided freeway section located at the north bound of the city of Athens. The section of a 4.7km length lies between the interchanges of Agios Stefanos and Varibobi and consists of the last part of the freeway before it penetrates into the city network. Its end was located (at the time of experimentation) 2.5km far from a signalized intersection (that of Kifisia). During the measurements the traffic conditions were not affected at any moment by the queue formed upstream the intersection, which never extended farther than one kilometer.

Tables 5-6 present data of the middle and left lane collected on a Sunday, the 21st of April 2002. Six experiments are presented, while the observer in three moved in the right lane and three in the middle lane. Those in the shoulder lane started at 18:02, 19:13 and 19:44 respectively, while the observer ran at 63.6km/h, 63km/h and 64.2km/h. Those in the middle lane started at 18:24, 19:30 and 19:58 respectively, while the observer ran at 68.5km/h, 67.7km/h and 77.6km/h. The prevailing traffic conditions in the specific section of the freeway in general are dense but rarely falling in congestion due to travelers who return home after a weekend (at the time of experimentation no fixed detectors were in use in the area for comparing their data with the measured data). In free flow conditions the average speed at the left lane is observed around 120km/h and at the middle lane between 90km/h and 100km/h. The speed that the observer developed during the experiments 70-80km/h and 65km/h in the middle and the right lane lanes respectively indicates that the traffic approached the onset of the congestion forcing the speeds to lower and the lanes to be synchronized. Therefore, the experimental results illustrated in tables 5 and 6, which provided speeds of 100km/h and 90km/h for the left and the middle lane and significantly high flows, were expected and in general justified by the traffic conditions.

The reproduction of normal speed distributions has not been statistically accepted in all cases (at a level of significance of 0.05). The speed distributions of both lanes are skewed towards the lower speeds. The reason is that there always appear some unusually fast vehicles that broaden its shape at high speeds, while the great mass of traffic moves with similar speeds narrowing its shape from the side of the lower speeds. Some frequencies close to the speed of the observer (right tail of the distribution) were considerably magnified disturbing the normally distribution shape. This magnification may be confined when the observer's speed differs for more than 20km/h from the slower speed class (i.e. the experiments for the left lane). Additionally, some frequencies close to the mean are lower than expected. The latter may be attributed to traffic fluctuations. However, it may be also the result of assigning rounded up numbers to narrow frequency categories (5km/h) of small magnitude. The data analysis consists of the following steps. First, all detected data have been used to predict the EN(predicted) data and their arithmetic mean speed and flow have been calculated using equations 10 and 11. These calculations represent the fundamental parameters describing the traffic conditions in a large space-time domain (Tables 5 and 6). Secondly, the same procedure has been conducted for a constricted space-time domain (around 1min x 1km), which is comparable to domains within which the fixed detectors function. Thirdly, the vehicles of each speed class which met the observer downstream, in the remaining section of 4.7km, have been identified. These vehicles EN(detected) met the observer up to varying distances depending on the speed of each class. Then the summation P+EN(detected), acting as virtual detector, might be equal to the counts of a fixed-detector at the beginning of the section. Because there wasn’t such a detector situated at that location at the period of measurement, that summation was used for evaluating the performance of the method in

- 22 -
both a short distance (1km) or long distance (4.7km). First, it appears that the dense conditions of high flow and moderate speeds are reproduced for the enlarged space-time domain in all cases (Tables 5 and 6). Contrary for constricted space-time domain, both the speed and the flow of the middle lane wasn’t successfully determined mainly because the observer moved at a speed only few km/h less that the detected vehicles. The determinations are improved in the case of left lane exactly because the speed difference between the observer and the detected vehicles is larger. The strong oscillation of flow in very short periods of measurements (i.e. 1min) affects the fixed detectors function as well. Moreover, the moving method attempting to predict the flow from significantly fewer data (only the P vehicles), when both the time of observation and the length of the section are short, fails to carry out the determination. However, a special attention may be paid to Figure 6. The average speed of the P vehicles in the entire section seems to considerably diverge from the average speed of those vehicles in the first-kilometre- section. Probably this is the outcome that the vehicles immediately after an onramp still don’t develop a high speed, while at the vicinity of the onramp they slightly slow down. Therefore, the experimental results of the second-kilometre-section may be more reliable of that of the previous kilometre-section, which may assume that the method could provide better results for intermediate sections. Based on this observation all calculations have been conducted for all experiments, which however hasn’t justified a definitive conclusion. Oppositely the method standing-alone seems to work for large space-time domains. In all cases the provided values of the fundamental variables are similar and comparable to the virtual detector.

7.2 Experimental work for recognizing changes of congested traffic conditions

The basic purpose of the experiment presented herein is to demonstrate the capability of the method to recognize and localize (spatially locate) traffic changes along a road section.

Data have been collected from a second freeway section. The section is located at the west bound of Athens (40-60kms away from it) and is a part of the national road of Athens-Korinth. It lies along three interchanges that of Agioi Theodori, Kineta and Megara. The section is initially a 6-lane divided freeway for 9.5kms (7km is the distance between the two former interchanges).

| Space-time domain | Speed class (km/h) | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 105 | 110 | 115 | 120 | 125 | 130 | 135 | 140 | Number of vehicles | Flow (veh/h) | Mean space speed (km/h) |
|-------------------|-------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|-----------------|--------------|------------------------|
| 1st experiment    | The observer moved at 64km/h in the right lane recording vehicles moving in the Middle Lane |
| 4.7km             |                   |    |    |    |    |    |    |    |    |    |    |    |    |    |    |               |               |                        |
| P(detected)       |                   | 4  | 6  | 6  | 5  | 4  | 4  | 4  | 2  | 2  | 43 | 4.3min        | 4.7km         |
| EN(predicted)     |                   | 12 | 14 | 13 | 12 | 8  | 6  | 5  | 3  | 2  | 92 | 1830          | 94            |
| EN(detected)      |                   | 2  | 3  | 2  | 1  | 1  | 1  | 1  | 1  | 1  | 2  | 1860          | 88            |
| EN+EN(detected)   |                   | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 70 | 1924          | 97            |
| 5th experiment    | The observer moved at 64km/h in the right lane recording vehicles moving in the Middle Lane |
| 4.7km             |                   |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |               |               |                        |
| P(detected)       |                   | 2  | 4  | 3  | 3  | 4  | 3  | 2  | 1  | 1  | 56 | 4.3min        | 4.7km         |
| EN(predicted)     |                   | 12 | 3  | 5  | 3  | 2  | 2  | 2  | 1  | 1  | 91 | 1790          | 88            |
| EN(detected)      |                   | 7  | 3  | 2  | 3  | 2  | 2  | 2  | 1  | 1  | 40 | 1920          | 87            |
| EN+EN(detected)   |                   | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 20 | 1960          | 85            |
| 5th experiment    | The observer moved at 64km/h in the right lane recording vehicles moving in the Middle Lane |
| 4.7km             |                   |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |               |               |                        |
| P(detected)       |                   | 2  | 1  | 4  | 4  | 3  | 1  | 1  | 1  | 1  | 73 | 4.3min        | 4.7km         |
| EN(predicted)     |                   | 12 | 3  | 5  | 3  | 2  | 2  | 2  | 1  | 1  | 56 | 1820          | 85            |
| EN(detected)      |                   | 7  | 3  | 2  | 3  | 2  | 2  | 2  | 1  | 1  | 40 | 1920          | 87            |
| EN+EN(detected)   |                   | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 20 | 1960          | 85            |
| 5th experiment    | The observer moved at 64km/h in the right lane recording vehicles moving in the Middle Lane |
| 4.7km             |                   |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |               |               |                        |
| P(detected)       |                   | 2  | 1  | 4  | 4  | 3  | 1  | 1  | 1  | 1  | 73 | 4.3min        | 4.7km         |
| EN(predicted)     |                   | 12 | 3  | 5  | 3  | 2  | 2  | 2  | 1  | 1  | 56 | 1820          | 85            |
| EN(detected)      |                   | 7  | 3  | 2  | 3  | 2  | 2  | 2  | 1  | 1  | 40 | 1920          | 87            |
| EN+EN(detected)   |                   | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 1  | 20 | 1960          | 85            |
Tab. 6 – Detected and predicted data of the 2nd, 4th and 6th experiments. Determination of mean space speed and average flow of various parts of the stream

<table>
<thead>
<tr>
<th>Speed class (km/h)</th>
<th>75</th>
<th>80</th>
<th>85</th>
<th>90</th>
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<th>100</th>
<th>105</th>
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<th>115</th>
<th>120</th>
<th>125</th>
<th>130</th>
<th>135</th>
<th>140</th>
<th>Number of vehicles</th>
<th>Mean space speed (km/h)</th>
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<td>2nd experiment.</td>
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<td>Detection</td>
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<td>Detection</td>
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<td>6th experiment.</td>
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<td>Detection</td>
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</table>

Fig. 6 – Successive speeds of vehicles met the observer during the 1st experiment

Two and a half kilometers farther than the second interchange towards the city, a bottleneck starts transforming the freeway to a 4-lane divided one along a distance of 8.5kms and with an intensively variable horizontal alignment. Finally, a third part consists again of a 6-lane freeway but with a significant downgrade (5%) in the city direction, which results to an increase of speed of passenger cars. The traffic inwards to the city usually reaches very dense conditions (sometimes congestion) during Sunday afternoons when weekend travellers return home. At that time an additional lane of the opposite direction is provided to users in the 4-lane subsection. Thus, the dense traffic travels in 2+1 lanes, while the outwards traffic uses the remaining one lane. Figure 7 contains data collected on this section indicating the way that the method could monitor traffic changes (the observer travelled in the right lane). The left part of the figure illustrates the stability of the speed at the 6-lane subsection. Then a gradual decrease of both the speeds of vehicles in the observing lane and the observer’s vehicle, however of different scale, shows that the denser conditions prevail.
At the middle lane, probably as the result of the split of its flow to this lane and the additional one at the opposite direction, the emergence of dense traffic is accelerated. Drivers may be indecisive as to which lane to follow producing a retardation of the flow. In the 2-lane subsection a stop-and-go traffic prevails with the speed collapsing in both lanes three times; however, it seems that the two lanes are not synchronized. Then speeds gradually increase showing that the flow after some time and distance recovers in stable conditions utilizing the available capacity. Finally, the motion in the last subsection is accelerated significantly due to the greater capacity and the downgrade. An interesting outcome is that all different conditions are localized in space and time, which could be important information about the evolution of the phenomena.

8. Conclusions

The traditional moving observer method determines the average flow and travel time of a traffic stream with limited accuracy and only for long time periods. The data collected may be simple, yet the method fails to acquire information about the prevailing speed distribution, which could control the quality of the estimates. Thus, the method is rarely used for routine measurements. However, in recent years, increasing use is being made of probe vehicles continuously moving in the highways, in order to support traffic management and information systems. These vehicles, in general, are assigned to measure the travel time, which is considered as a basic feature to describe traffic conditions. The present work revisits the method with the objective to enrich it methodologically, arguing that its quality can be considerably improved when the speed and the length of all recorded vehicles are measured instead of their cumulative number (against-the-stream technique) or ‘tally’ number (with-the-stream technique). In this way the speed distribution could be determined, based on which the flow, the mean space speed of the stream and its traffic synthesis could be calculated in non-congested traffic conditions. This becomes possible through the installation on the observing vehicle of a system consisting of two (or four) detectors and a GPS-based subsystem. The observer moves in one lane and records the vehicles moving in an adjacent lane. The detectors function similarly to a double presence-type detector, while the GPS provides absolute positioning and timing of any event of interest (activation and deactivation of detectors, locations of traffic changes etc.). Three techniques have
been developed, the 1st being powerful to determine simultaneously the fundamental variables in two or three lanes, while the other two in each separate lane of the freeway section. Although the method standing alone estimates the flow and the mean space speed for relatively long freeway section, the exact space speed distribution could be determined when combined fixed-detector and moving measurements are used. The determination of speed of all vehicles met by the observer is possible in congested conditions as well. In this case, the observer could follow the stream fluctuations of his/her adjacent lane, especially when non-synchronized lanes and stop-and-go traffic prevail. Experimental work proved the capability of the 2nd technique to collect data in separate lanes of a 6-lane freeway, and provide the average flow and the mean space speed for short time periods and enlarged space-time domains. Nevertheless, it revealed its weaknesses when the observer’s speed is selected close to the slower speed classes or when it is used for constricted space-time domains. Finally, it was shown that the method could recognize differences in traffic conditions (congested and non-congested) along a long highway section, similarly to what a driver perceives in the vicinity of his/her motion.

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References
Evaluation of a Behavioral Cellular-Based Traffic Model

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Abstract

A cellular-based model that improves cell-based models’ ability to replicate actual traffic flow on high speed highways is proposed in this study. Behavior variations were introduced in the model by incorporating both driver aggression and responsiveness to the action of the leading vehicle. The deceleration and acceleration of the following vehicles were based on their speed and distances relative to leading vehicles. The evaluation of the model performance indicated that the proposed model was able to reproduce traffic conditions observed in the field at an acceptable level of accuracy as a MAPE statistic of 9.9% was obtained. A low (0.002) Theil’s inequality coefficient also confirmed the validity of the model specifications. Correlations of average speeds and traffic flow from the simulation and actual traffic data verified model’s output match real world data. Further examination of individual vehicle trajectories from simulation results showed that the model can reproduce backward forming traffic jams consistent with empirical observations.

Keywords – cell-based micro simulation, cellular automata, traffic modeling

1. Introduction

Cell-based traffic simulations rely on global consequences of local vehicle interactions in the traffic stream. In the past decade, cell-based traffic simulation models have emerged as an alternative tool for investigating traffic flow phenomenon [6,4,19]. One kind of these models is the cellular automata (CA) traffic flow model which uses discrete space and time scales which make it simple to understand and fast to simulate. The CA models typically consist of a framework where vehicle interactions occur. The movements of vehicles in these models are performed through sets of procedural rules which depend on speeds and locations of neighboring vehicles as well as characteristics of the road. Despite encouraging results reported from the application of CA models in modeling both one-dimensional [e.g., 14] and two-dimensional [e.g., 21] traffic flow, they have some deficiencies that are now widely recognized. The deficiencies include lack of behavioral variation in the model, unbound deceleration rates, and large positional updates which are independent of the vehicle initial speeds. Thus, to improve the ability of these models to simulate traffic systems more accurately, there is a need to capture the variability of
traffic flow caused by actual driver behavior. In the context explained above, this paper proposed a model that incorporates vehicle kinematics and driver behavior into the CA traffic model. These improvements are expected to improve the CA models’ ability to replicate traffic behavior on high speed highways. The movements of individual vehicles are modeled using one-dimensional kinematic equations for constant acceleration motion. Any forces that cause the vehicle motion to change laterally on its lane during longitudinal motion are ignored. Driver behavior is modeled by the reaction of the driver to the prevailing speeds and space headways. The introduction of driver behavior and kinematic equations in the model is expected to supersede some of the limitations found in the conventional CA models. The introduction of driver behavior in the proposed model involves applying both aggression and alertness or responsiveness of the driver in reacting to the prevailing traffic conditions. In addition, the proposed model has incorporated mechanical restrictions due to vehicle dynamics through the application of different maximum acceleration and emergency deceleration rates for truck and passenger vehicles. This paper has two objectives: (1) to develop a cell-based model that addresses vehicle and driver limitations identified in the existing CA models, and (2) to compare the proposed model to the observed vehicle trajectories data collected from a section of a freeway and evaluate its ability to predict speed and spacing profiles over time. The rest of the paper is organized as follows: The next section provides a brief review of the existing CA models which is followed by the section that discusses the establishment of the proposed model, its underlying concept and modeling approach. Then, the testing and evaluation results of the proposed model are presented before discuss conclusions and direction for future research in the last section.

2. Review of CA traffic models

Cellular automaton (CA) is the mathematical representation of a physical system in which both space and time are discrete. In this representation physical quantities are taken on finite sets of discrete values. A CA model consists of a regular uniform and finite lattice that takes discrete variables which occupy different sites. The traffic flow problem is formulated in terms of cellular automaton by modeling individual vehicle movements in a discrete time scale based on the amount of empty cells (gap) ahead. Once the amount of gap is determined, a simple rule is used to update the speed and hence the position of the subject vehicle.

The use of CA in traffic flow simulation is based on the elementary cellular automaton rule 184 proposed in the Wolfram classification [23]. Application of rule 184 to the traffic flow modeling was made possible by representing the highway as the lattice of cells which can either be empty or occupied by vehicles. Each cell corresponds to a road section with a uniform length equal to the vehicle length. Using this rule, vehicles can move forward whenever there is an empty cell ahead. Therefore, all cells that are occupied by vehicles are assigned “1” and empty cells are assigned “0”. This rule is also known as deterministic CA traffic rule because the evolution of the bits is for the “1” to occupy the neighboring “0” during simulation. Eventually, rule 184 prevents two vehicles from occupying the same cell. It is worthwhile noting that this rule prevents the vehicle behind from moving forward even when the leading vehicle moves forward during the same iteration. Another interesting property of the CA traffic model is that the number of occupied cells corresponds to the number of vehicles. The maximum speed attained by vehicles in this model is 1 cell per unit time step.

Nagel and Schreckenberg [19] were among the first to recognize the capability of the Wolfram rule 184 for traffic flow micro simulation. The authors modified this rule to enable vehicles to evolve with different velocities during simulation in a ring-like road with closed boundary
conditions. The fundamental idea behind the CA traffic micro simulation by the authors was to formulate the model in discrete time and space, in favor of numerical efficiency. In the Nagel-Schreckenberg model, driver-vehicle units are identical and their states at each simulation time step are determined by their respective instantaneous velocities and positions. The basic vehicle position update rules in the CA traffic model suggested by Nagel and Schreckenberg [19] are as follows:

(i) The vehicle accelerates if its velocity is lower than the maximum speed and if the distance to the vehicle ahead is larger than its current speed plus one.
(ii) If the distance of the subject vehicle to the vehicle ahead is less than its current speed, then the vehicle reduces its speed to the amount of available distance.
(iii) With a certain predefined probability, the speed of each vehicle (if greater than zero) is randomly reduced by one.
(iv) Each vehicle is advanced ahead by amount of cells equal to the velocity obtained in (iii).

Rules (i) and (ii) define the optimal driving strategy of the drivers depending on the prevailing traffic conditions. However, due to some latent factors which are not accounted for in Rules (i) and (ii), driving behavior may vary resulting into a stochastic behavior of traffic. This is accounted for in Rule (iii). In addition, this rule is responsible for spontaneous jam formations in the simulation. The CA model was tested on closed boundary conditions and the simulation results replicated the basic features of the real traffic flow such as fundamental correlations between vehicle speeds and traffic density as well as the formation of traffic jams. This model is sometimes referred to as the basic CA traffic model. The CA model has been applied in simulation of road networks after the set of fundamental rules described above were expanded to include lane changing and route decision rules. For example, the concept of Nagel-Schreckenberg model was applied to develop transportation analysis and simulation system software (TRANSIMS) by Los Alamo research laboratory and other simulation tools that simulated large scale urban networks around the world [20,21,10].

Since the development of Nagel and Schreckenberg model, many forms of CA traffic models have been proposed. Fukui and Ishibashi [8] proposed a model which employed abrupt increase in acceleration whenever there are enough space gaps ahead of the subject vehicle. The Fukui and Ishibashi model resembles that proposed by Nagel and Schreckenberg when the maximum velocity (vmax) is 1 cell per time step. Other recent developments of Nagel-Schreckenberg model, include improvement of the CA traffic model by incorporating velocity-dependent randomization component and driving anticipation [e.g. 16,14], and slow to start rule [2]. Also, Knospe et al. [13] proposed a CA model which considers the desire of the driver to attain smooth and comfortable driving. All these improvements have focused on reducing erratic traffic behavior found in the conventional CA models. Some advantages of CA traffic models are related to their use of very simple vehicle position update rules and quick realization of numerical simulation due to the use of discrete variables. In addition, application of the CA models into parallel computers is easily applicable. One major drawback of these models is that all driver and driver entities inside the model have the same physical and behavioral characteristics. However, some CA models have used different speed limits for trucks and passenger cars [e.g. 13]. Recently, Lee et al. [15] developed a CA model that simulated a collision-free movement by incorporating maximum braking capacity and human overreaction in terms of optimal or defensive driving strategies. This was a significant attempt towards reducing erratic decelerations in the CA traffic modeling.
3. Proposed kinematic cell-based model

One dimensional kinematic cell-based model presented in this paper was initially tested on single-lane of high speed traffic [18]. The model was able to reproduce realistic traffic characteristics in both dense and light traffic conditions. Special features of the model included the ability of vehicles to occupy multiple cells and the use of kinematic equations to model vehicle movements. The kinematical CA model has bounded deceleration rates that reduced erratic vehicle movements found in the existing CA models. Besides extending the kinematic modeling of two-lane traffic, the new proposed model presented in this paper comprises two vehicle types—passenger vehicles and trucks differentiated by length and acceleration capabilities. Also, the driver type has been introduced by incorporating driver aggression and alertness in the model. The incorporation of different vehicle and driver types in the model was designed to increase the ability of the kinematic CA model in replicating real world traffic flow variations. The inclusion of driver behavior in the model is also expected to increase the usefulness of the proposed model as a predictive tool in assessing safety measures at a microscopic level.

3.1 Model Structure

A schematic conceptual model proposed in this study is summarized in Figure 1. The modeling controlled number of lanes, roadway alignment, grade, and roadway conditions. A two-lane roadway was discretized into 2-ft long cells. The roadway has beginning and ending nodes as well as exit and entrance nodes where vehicles can enter and leave the roadway. No acceleration/deceleration lanes were used for on/off ramp traffic. Drivers and vehicles were modeled as single objects—that is, each generated vehicle was assumed to have both driver and vehicle attributes. The inputs were the initial speed, desired speed, arrival rate, and vehicle and driver compositions. With the exception of parameters that described distribution of drive-vehicle objects, other input parameters were integers. To reduce rapid initial decelerations, the entrance speed of the vehicle in the simulation was set to be the minimum of the initial speed of the vehicle generated and the spacing headway between the entering vehicle and the leading vehicle already in the system.

Fig. 1 – Conceptual traffic simulation model.
Randomness was introduced and maintained throughout the simulation by generating driver-vehicle parameters using random numbers. The computer (system) time was used to generate different random seeds in order to obtain different pseudorandom numbers. Each vehicle-driver object was assigned maximum acceleration/deceleration, reaction time, and anticipation of the leading vehicle actions. The desired speed of the vehicle-driver entity was generated from a distribution of speeds of vehicles under low volume conditions (less than 600 vph). The simulated vehicles were moved with a predefined vehicle movement logic that uses kinematic equations and maximum acceleration/deceleration of the vehicles. The movement of the vehicle was affected by the relative speed and the space headway. Errors caused by driver limitations in estimating vehicle speeds led to the introduction of a random noise designed to increase or decrease the acceleration of some of the simulated vehicles. The random noise was introduced in such a way that some aggressive drivers were assumed to increase speed by 2 cells per second while some non-aggressive drivers reduced their speed by the same amount. This randomization of individual vehicle is a paradigm shift from CA traffic models which decrease speed of all vehicles during a single simulation time step.

3.2 Driver Motivation

Aggressive driver behavior was introduced in the model based on driver risk-taking concept suggested by Berkowitz [3]. In this concept, an aggressive driver puts other drivers in danger without intending to harm them. Thus, the modeling used two key factors in predicting risk-taking behaviors: the intent to arrive sooner and the position that the driver is willing to take in the traffic stream. While the later affects the headway that the driver maintains in car-following situations, the former affects the driver maximum desired speed. Based on the desired speed distribution, the driver aggressiveness was divided into three levels with the most aggressive driver being at the higher end of the maximum desired speed distribution while the cautious driver was at the lower end. The preferred time headways were assigned to the aggression spectrum with very aggressive drivers assigned lower preferred time headways. However, the modeling allowed the moderate aggressive drivers to become aggressive when following slow moving vehicle in which they are forced to change lane so as to increase travel speeds.

Since attentiveness and alertness of a driver contributes to the variation of traffic flow, another parameter was introduced in the model to simulate driver attentiveness. It is worth mentioning that previous research results have shown that driver’s inattention and delayed reaction have a negative influence on safety [7]. Therefore, in the model proposed in this study, a delayed reaction time was introduced for some drivers who have a high degree of inattention or are slow to respond to the action of the leading vehicle. The alertness level of these drivers was termed ‘poor’. The drivers with “poor” alertness demonstrated inconsistencies in estimating leading vehicle speeds and were also slow to respond to leading vehicle actions.

3.3 Vehicle Movement Rule

In the proposed model, the initial speed and the amount of acceleration determined the movement of vehicles in the simulation. The amount of acceleration was estimated based on relative speed and the space headway between the subject vehicle and the leading vehicle. This assumption conforms to many stimulus-response car-following models [17] but in this model it is being used from a cause-effect perspective. Many stimulus-response car-following models use empirical based mathematic equations to model vehicle acceleration. In this study, the
acceleration of vehicles is modeled using a deterministic rule-based algorithm that is summarized in Table 1 and implemented in the acceleration logic function using IF-THEN clauses. The acceleration logic in the model is divided into three different states, namely free-flowing, dense, and emergency. When developing the rule-based algorithm for vehicle acceleration, the following actual movements of the vehicle were taken into consideration: accelerating, cruising, decelerating or idling depending on the prevailing traffic state. Their corresponding driver actions are acceleration, cruising, deceleration, or rapid deceleration. Inside the model, the driver adapts to the existing condition by accelerating, decelerating, or maintaining a constant speed after evaluating the existing space headway and relative speed.

The new speed $v_{i,new}$ of the subject vehicle, $i$, after it has accelerated at a constant acceleration, $a_i$, during time $t$ from an original speed, $v_{i,old}$, was updated using the following kinematic equation:

$$v_{i,new} = v_{i,old} + a_i t.$$  

This equation leads to a vehicle accelerating freely to the maximum desired speed unless constrained in its path.

Each vehicle type has two constant acceleration parameters—one at lower speed and the other at higher speed. In addition, passenger cars had higher acceleration rates than trucks.

The amount of deceleration, $d_i$, of the subject vehicle was computed using the following equation

$$v_j^2 = v_{i,old}^2 + 2d_i s_i$$

where $s_i$ is the available distance headway in the next time step. Note that deceleration and acceleration were used separately because reactions of the drivers when approaching relatively slow moving vehicles differ among drivers. The position of the vehicle $x_{i,new}$ is updated by using the average speed and the time elapsed as

$$x_{i,new} = x_{i,old} + \bar{v}_j \times t$$

where $\bar{v}_j$ is the average of the initial and final speeds of the vehicle. The rationale of using the average speed in computing new vehicle’s position is to incorporate the rectilinear motion and hence minimize over displacement of vehicles in the system. For a conflict free movement, the necessary condition is that the distance traveled by the subject vehicle, $x_i$, should be less or equal to the sum of the available clearance gap and the distance that the leader will travel in the same time step, $x_{i-1}$. That is, $x_i \leq x_{i-1} + (gap - g')$. The available spacing is the clearance distance between the subject vehicle and the leading vehicle less the minimum clearance, $g'$, that the driver of the subject vehicle is willing to maintain during congested traffic conditions. The minimum gaps were different for different pairs of vehicle types. Gipps lane-changing rule was implemented [9]. It is a rule-based algorithm with three steps. First, the driver checks the need to change lane and the type of change needed to be executed.

<table>
<thead>
<tr>
<th>Traffic State</th>
<th>Description</th>
<th>Subject driver’s action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free-flowing</td>
<td>Space headway is greater than 300 ft; leading vehicle actions do not influence the subject vehicle.</td>
<td>• Accelerates to reach the desired speed.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Cruises at the desired speed.</td>
</tr>
<tr>
<td>Dense traffic</td>
<td>Approaching a relatively slow moving vehicle.</td>
<td>• Decelerates so that the relative speed is zero and seeks to maintain the desired following distance.</td>
</tr>
<tr>
<td>Following the leading vehicle subject vehicle has a lower speed.</td>
<td></td>
<td>• Accelerates if the spacing is larger than desired spacing.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Driver cruises at the desired spacing.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• The relative speed oscillates around zero.</td>
</tr>
<tr>
<td>Emergency situation</td>
<td>Traffic in bumper to bumper movements.</td>
<td>• Accelerates/stop to maintain minimum space headway.</td>
</tr>
<tr>
<td></td>
<td>The leading vehicle suddenly decelerates.</td>
<td>• Driver applies high deceleration rate to avoid collision.</td>
</tr>
</tbody>
</table>
Second, the driver selects the target lane; and lastly, the driver executes the desired lane change whenever there is an adequate gap. Considerations were given to two types of lane changing movements—i.e. discretionary and mandatory lane changing. Discretionary lane changing was assumed when the driver is approaching a slow moving vehicle and changing of the lane is sought to increase speed to reach the maximum desired speed. Mandatory lane changing occurred when the driver was required to leave the inside lane to take the off ramp located on the outside lane. In discretionary lane changing, the decision to change lanes is based on the speed of the leading vehicle and the gap available in order to safely move to the desired lane. The driver’s decision to change lane in the proposed model is affected by prevailing traffic conditions in both current and target lanes. First, the relative speed of the subject vehicle with respect to the leading vehicle is calculated. If the relative speed is positive (subject vehicle is moving faster than leading vehicle) then the subject vehicle is allowed to change lane whenever there is adequate gaps. In addition, the vehicles are allowed to yield to the exiting vehicles that performed mandatory lane change by either changing lane to the left or reducing speed to avoid collisions. The incorporation of courtesy yield factor was design to improve the simulation to reflect real life driving behavior.

Another lane changing capability allowed in the model was shifting the exit destination of the vehicles that were unable to successfully perform mandatory lane change to exit the roadway. When vehicles encountered such situations in the simulation, their destinations were changed to the nearest exit point.

4. Model evaluation

The kinematical CA model proposed in this study was implemented in visual C++ programming. The process of evaluating the accuracy of the model in replicating real world traffic conditions involved two processes—calibration of microscopic characteristics and examination of macroscopic characteristics. The calibration process involved tuning and adjusting the input parameters to optimize the accuracy of simulation model results as compared to actual traffic conditions. The parameters that required calibration were those related to traffic control operations, traffic flow characteristics, and drivers’ behavior. The examination of the macroscopic characteristics was done by analyzing correlations between average speed, density, and traffic flow statistics collected from virtual detection points set along the simulated roadway.

4.1 Simulation experiment

A two-mile long, two-lane highway with unidirectional traffic was used to calibrate the model. The roadway was made up of 5,280 cells each 2 ft (0.6 m) long. The use of small cells was necessary to discretize the speed in multiples of 2 ft/s (0.6 m/s). This enabled collection of many speed bins from the simulation output and smaller acceleration rates. Besides the beginning and ending nodes, the roadway had exit and entrance nodes. At the beginning of the road, the vehicle could enter the roadway in each lane based on a predefined arrival rate. In addition to the beginning and ending nodes, vehicles entered the road at on-ramps located at the 1420th and 4060th cells; and left the road at off-ramps located at the 1620th and 4260th cells. As soon as a vehicle reached its destination, it was removed from the system and its memory freed. Other parameters used in the simulation were as follows:

- When the speed was less than 50 mph (80 kph), maximum acceleration rates for passenger cars and trucks were set at 8 ft/s² (2.4 m/s²) and 6 ft/s² (1.8 m/s²), respectively, otherwise 6 ft/sec² (1.8 m/s²) and 4 ft/sec² (1.2 m/s²) were used, respectively.
- Maximum normal deceleration rates was 16 ft/s² (4.9 m/s²).
- Maximum allowable emergency deceleration rate of 32 ft/s² (9.7 m/s²) and 24 ft/s² (7.3 m/s²) for cars and trucks, respectively were specified.
- Cautious, normal, and aggressive drivers had 3 s, 2 s, and 0.9 s preferred headways, respectively.
- 20% of the drivers had ‘poor’ alert characteristics.
- Free-flow was assumed when a vehicle has a clearance gap of at least 300 ft.
- In standstill conditions, minimum spacing of 20 ft (6m) between two passenger cars, 60 ft (18 m) between passenger car and a truck, 40 ft (12 m) between truck and passenger car, and 40 ft between two trucks were used.
- Average lengths of 30 ft (9.1 m) and 12 ft (3.6 m) for trucks and passenger cars, respectively were specified.
- 10% of the vehicles were trucks.
- Probability that a random noise will be introduced in the model was 20%.

Each simulation scenario was run several times with different random number seeds. Since the proposed model is stochastic, multiple simulation replications were conducted and average values of the output were used to assess its performance. The number of simulation replications was estimated based on a method presented by Chu et al. [5]. From the pilot simulation, the number of replications, N, was estimated as follows:

\[
N = \left( \frac{t_{\alpha/2,n-1}}{\delta/\mu} \right)^2
\]

where \(\mu\) and \(\sigma\) are the mean and standard deviation of the performance measure obtained from the pilot run, \(\varepsilon\) is the allowable error specified as a percentage of the mean—normally it is half length of the confidence interval; \(t_{\alpha/2}\) is the t-statistic of the t-distribution at \(\alpha\) % significant level. Using a 10% significant level and a 5% allowable error, the number of runs based on the sample variance of spacing and speeds obtained from a pilot simulation of 10 replications suggested the need for approximately 22 runs per simulation scenario. Since the simulation results had a wide range of variations, the number of simulation runs per replication was increased to 25 in order to increase the confidence of the results.

At each time step, individual vehicle information were collected from virtual detection points and stored. The averages of the individual vehicle information, collected at virtual detection points, were later used to derive macroscopic statistics such as average speed, flow, and density. Thirty seconds interval of sampling was used to calculate macroscopic characteristics. Due to the nature of the cell-based approach, the occupancy data were calculated in form of the density which was obtained as the reciprocal of the mean space headway.

### 4.2 Quality control checks

Checks were performed to control quality to ensure that the coding and algorithm used in the programming did not have any amplification or coding errors. Amplification errors are common errors in the input data that can skew the results. Once the model was deemed error-free, its output was verified by comparing the desired speeds of individual vehicles to their corresponding average travel speeds. First, the speeds of the vehicles were examined under low traffic conditions and then under progressively increasing traffic volume. Figure 2 depicts the relationship between desired speeds and average speeds under low and high traffic conditions.
Low volume traffic

Heavy volume traffic

Fig. 9 – Desired speed versus average travel speed

On the left of Figure 2, it is seen that the model predicts the travel speed relatively well under very low traffic conditions, i.e., specified as 360 vehicles per hour per lane. In these conditions, the vehicle interactions are minimal and therefore vehicles were traveling at their desired speeds, in most of the times. The diagonal line is the theoretical relationship between the two speed values. The data points in some sense deviated from the theoretical line because of the randomness introduced in the model. The right side of Figure 2 shows the relationship between the two speeds under heavy volume conditions, specified as by an average arrival rate of 1,800 vehicles per hour per lane. In this case, due to the constraints introduced by on and off ramp traffic as well as slow moving vehicles presented in the system, the average speeds of the vehicles were lower than their corresponding desired speeds—neither vehicle attained its maximum desired speed. Following this verification, the model was calibrated using actual traffic data.

4.3 Calibrating driving behavior

Field data used for validating the performance of the proposed model were obtained from the Next Generation Simulation (NGSIM) database [1]. This database contained vehicle trajectories data collected on April 2005 on Interstate 80 section in California between 5:00 to 5:15 PM. The Interstate 80 data used for validation consisted of records of local positions (lateral and longitudinal coordinates) of individual vehicles as well as other parameters describing vehicle types, the identifications of the preceding and the following vehicle. In addition, 30-second detector data which show average speed, occupancy, and volume information were also useful for model evaluation. All vehicles that were in a car-following situation in the innermost lane were extracted from the database and exported to statistical software for analysis. Only the inner most inside lane was used to query vehicles in no-lane change situation because many vehicles traveling in this lane did not change lane frequently. The attributes of interest from the extracted vehicle data were the longitudinal position, speed, preceding and following vehicles, spacing and time headway. Data in the innermost lane were queried from the database and exported to statistical software where they were prepared for use.

Consistent with the data available in the data base, simulated vehicle trajectories data were also collected along a 1,500 ft (457.2-m) section for a 15-minute period. The model was calibrated and its validity was checked using three statistical measures: Pearson’s coefficient of
correlation \((r)\), Theil’s inequality coefficient \((U)\), and mean absolute percentage error \((MAPE)\) as shown in the following equations:

\[
r = \frac{1}{n-1} \sum_{i=1}^{n} \frac{(y_{i,\text{sim}} - \bar{y}_{\text{sim}})(y_{i,\text{obs}} - \bar{y}_{\text{obs}})}{\sigma_{\text{sim}} \sigma_{\text{obs}}}
\]

\[
U = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (y_{i,\text{sim}} - y_{i,\text{obs}})^2}
\]

\[
MAPE = \frac{1}{n} \sum_{i=1}^{n} \left| \frac{y_{i,\text{sim}} - y_{i,\text{obs}}}{y_{i,\text{obs}}} \right| \times 100
\]

where \(n\) is the total number of data points, \(y_{i,\text{sim}}\) is the \(i^{th}\) simulated data point, \(y_{i,\text{obs}}\) is the \(i^{th}\) observed data point, \(\bar{y}_{\text{sim}}\) and \(\bar{y}_{\text{obs}}\) are the means of the simulation output and actual values observed in the field, respectively. \(\sigma_{\text{sim}}\) and \(\sigma_{\text{obs}}\) are the standard deviations of the simulated and observed values, respectively. The \(r\)-statistic of -1 indicates a strong negative linear relationship while a value of 1 signifies a strong positive relationship. The numerator of \(U\) is the root mean square error (RMSE). This statistic provides additional information about the nature of the error between simulated and actual data. \(U\) has the lower limit of zero which signifies a perfect fit lack of fit. \(MAPE\) is the statistic which measures the within simulation goodness-of-fit and out-of-simulation performance. It is calculated as the average of the unsigned percentage errors for data that are strictly known to be positive. It has lower and upper limits of zero and 100 which indicate perfect and poor fit, respectively.

A major calibration issue in this study concerned the relationship between individual vehicle speeds and their respective space headways. Since the acceleration model depends on the individual speeds of the subject and the leading vehicles as well as the relative spacing the relationship between speed and spacing was important in calibrating the model. The mean spacing for each speed bin of the actual traffic data were calculated and compared with the simulated spacing for the same speed group. Starting with the initial parameters that were satisfied in the model verification stage, the model was simulated and spacings were compared with the actual field data extracted from NGSIM database. The process was continued by adjusting the model parameters until the performance evaluation criteria were met. The model performance results obtained after calibration are shown in Table 2.

Tab. 2 – Model performance evaluation

<table>
<thead>
<tr>
<th>Data</th>
<th>Performance evaluation criteria</th>
<th>r</th>
<th>U</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>5:00-5:15 p.m.</td>
<td></td>
<td>0.87</td>
<td>0.002</td>
<td>9.9</td>
</tr>
<tr>
<td>5:15-5:30 p.m.</td>
<td></td>
<td>0.63</td>
<td>0.003</td>
<td>12.4</td>
</tr>
</tbody>
</table>
Figure 3 shows the predicted spacings versus the observed spacings from the database. The calibration process adjusted driver aggressiveness parameter to produce the spacings presented on the left side of Figure 3. Data presented in Figure 3 revealed that actual traffic data had very high variations at higher speeds. The simulation results did not accurately catch actual speed-spacing variations observed in the field data at higher speeds. In addition, observed data showed that drivers take longer standstill distances than what was predicted by the model. Closer examination of the stopping vehicles in the actual traffic data showed that some of the vehicles stopped even with over 150 ft spacing. This necessitated the increase of minimum space headways during stop and go motion in the model specifications. However, the model performance did not improve much. Statistical comparison of the simulated spacing with observed spacing in 5:00-5:15 p.m. time period produced a MAPE statistic of 9.9%, which satisfied the calibration acceptance criteria of 15% set in this study. A low Theil’s inequality coefficient also confirmed the validity of the model specifications in predicting actual traffic flow. The simulation was later tested against data observed between 5:15-5:30 p.m. Statistical comparisons showed acceptable levels of the MAPE and Theil’s inequality coefficient. However, the comparison had a moderate correlation coefficient. Large variation of observed spacing under low and high speeds accounted for poor fit in the model.

4.4 Correlation between macroscopic characteristics

Correlations between space mean speed and traffic flow are shown in Figure 4. Figure 4 shows that both saturated and unsaturated traffic conditions were observed in the simulation. The generalized speed-flow relationship on freeways that was suggested by Hall et al. [11] and later adopted by the Highway Capacity Manual [22] seems to be replicated in Figure 4. The transition from uncongested to congested conditions is somewhat visible from the simulation results. Figure 4 shows that capacity is about 2,400 vehicles per hour per lane. Closer examination of Figure 4 further reveals widely scattered as well as variations of the traffic data which can indicate a degree of randomness of traffic flow in the simulation. Although no bottleneck (such as lane closure or construction) was introduced in the model, traffic congestion observed in the model which is evidenced by decrease of traffic speed and volume may be the result of vehicles entering at the ramps or lack of opportunity to pass slow moving vehicles in the model under heavy flow conditions.

Fig. 3 – Simulated vehicle spacing versus observed vehicle spacing
Further validation of the proposed model involved analyzing correlation between speed and traffic flow from the field data as presented in the right side of Figure 4. The comparison of field and simulated correlations shows that traffic flow characteristics obtained from the model agree with the ones observed in real world. Free-flow conditions were modeled by using low arrival rate while high arrival rates were used to trigger heavy traffic conditions. Field data showed that the maximum average free-flow speed of traffic was about 65 mph (105 kph) which is almost close to what was observed in the simulation results. Note that the desired speed distribution used in the simulation was the same as that observed in the field under low volume conditions (600 vphpln). Another point worthy of mentioning is that the maximum speed found in the simulation is closely the same as the one observed from actual data. However, the proposed model did not reproduce the exact shape shown in the right side of Figure 4 probably because the driving behavior in the simulation is more or less restricted to the logic and algorithm used which could have led to some deviation from the real life driving behavior, especially in the transition from free-flowing to congested traffic flow. Spatio-temporal variations of individual vehicles were later used to study the behavior of simulated traffic. At low volume conditions, the traffic was free flowing most of the time and no traffic jams were formed. Under these conditions, every vehicle-driver entity was moving at the desired maximum speed because there were sufficient gaps to pass in the adjacent lanes whenever a high speed driver encountered a low speed driver. Figure 5 shows time-space trajectories of vehicles under high volume flow. Back propagating jams can be seen in the simulation results in Figure 5.
The jams, indicated by dark spots in the figure, are places where a vehicle slowed down abruptly. A back propagating jam is an upstream movement of the slow traffic as the result of increased density downstream. It is characterized by a significant drop of speed. The velocity of the back propagating jam was estimated to be -16.5 ft/s (18.1 kph). This speed is close to a 13.7±4.6 ft/sec (15±5 kph) speed of backward congestion wave that has been observed in real life traffic flow in different countries [12]. Similar results were obtained in simulation studies [e.g. 15] Figure 5 indicates that vehicle moving in a jam have low speeds compared to vehicles traveling upstream and downstream of the jam. Some of the vehicles were also caught in stop-and-go mode. This is evidenced by the zero slopes (change in position per unit time) in the moving jam areas. As soon as the vehicles passed the moving jam they started accelerating to reach their desired speed.

5. Conclusions

This study proposes a behavioral cell-based traffic model to address limitations found in most cellular automata traffic flow models. The movements of individual vehicles were governed by one-dimensional kinematic equations with bounded acceleration and deceleration rates. The use of kinematic equations to model vehicle movements was designed to increase the fidelity of the model in replicating real life traffic conditions by reducing erratic decelerations found in the previous models. Lane-changing logic was implemented based on Gipps model [9]. The model proposed in this paper is part of the research on using cell-based micro simulations to identify safety indicators that can cause conflict and crashes so that safety measures can be instituted at the early stages of highway design using micro-simulation predictive methods.

The model validation showed that it performed well on high speed two-lane traffic with on-ramp traffic. The evaluation of the model was accomplished through calibration of driving behavior using real life traffic data. The calibrated model showed reasonable performance in replicating actual traffic flow. Comparing the space headways and speeds obtained from the model and vehicle trajectories data observed in the field returned a MAPE statistic of about 9.9%. Correlation studies also revealed that average speed, flow, and density from the model agree not only with theoretical studies but also with detector data obtained from the field. Examination of spatio-temporal variation of individual vehicles in the models showed the formation of traffic jams under heavy traffic flow. Back propagating jam with a speed of -16.5 ft/s (18.1 kph) was observed in the simulation.

The paper further addressed various traffic and driver behaviors to improve cell-based micro simulation. However, because of the geometric limitations—lack of curvature, grades, and acceleration/deceleration lanes—in this model it is difficult to accurately draw major conclusions regarding roadway capacity and other operational measures. The simulated highway did not have acceleration or deceleration lanes. The dynamics of traffic flow in these areas generally have an impact on safety and operations of a real life highway. One of the future areas of study related to this research is the development of a systematic microscopic tool that can simulate traffic using the movement logic proposed in this study while addressing these geometric limitations.

References


State-of-the-art intelligent road design model with genetic algorithms, geographic information systems, and CADD

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Abstract

A state-of-the-art intelligent road design model is developed that has the ability to simultaneously optimize 3-dimensional highway alignments, exploit Geographic Information System (GIS) maps and databases for enhanced practical applications, and view detailed design features, including road animation and digital terrain models. We provide an overview of the integration of the highway design procedure using an AutoCAD-based package called RD 2000 with a Highway Optimization Model (HAO) developed by our research team. The initial 3-D highway alignment optimization problem with genetic algorithms was studied by our research team in 1996; since then successive enhancements to the model has been made resulting in several real-world applications. The development of the intelligent road design model enables integration of CADD and digital terrain modeling capabilities to the developed genetic algorithms and GIS-based optimization model. An example from Maryland demonstrating full potential of the model is presented. Several future enhancements to the model are also discussed.

Keywords – Intelligent road design, highway alignment optimization, Genetic algorithms, GIS, CADD

1. Introduction

There are numerous alternatives that must be analyzed when planning and designing roads. Many complex and conflicting factors have to be considered. Among these factors include topography, geology, hydrology, land-use and values, environmental impacts, construction procedures and costs, traffic flows, safety, interfaces with present and future networks, life-cycle maintenance and user costs, and political concerns and preferences. Numerous uncertainties exist and poor location settings can occur when selecting transportation facilities. Billions of wasted dollars have resulted from last minute design changes and relocations. This is attributable to the complexity associated with manually attempting to optimize the locations of transportation facilities and lack of automated methods for this task. Some noted projects that have fallen into this persona of planned, ongoing, and recently completed/failed projects are shown in tables 1 and 2.
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Tab. 1 - Examples of major road improvement projects

<table>
<thead>
<tr>
<th>Project Description</th>
<th>Method used for selecting route</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Widening of 325 miles of Interstate 81 to separate car and truck lanes in Virginia, estimated cost: 6.3 billion</td>
<td>Engineering judgment, public/political input, and trial-error</td>
<td>Engineering News Record (ENR), March 1, 2004, p. 15.</td>
</tr>
</tbody>
</table>

Tab. 2 - Examples of major transportation failures

<table>
<thead>
<tr>
<th>Project Description</th>
<th>Probable Cause</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay in Las Vegas Strip’s high-speed monorail, $650 million, 3.8 mile, January 2004</td>
<td>Poor cost estimation, design and location analysis</td>
<td>ENR, June 28, 2004, p. 14</td>
</tr>
<tr>
<td>Design and Scope changes with Boston’s Center Artery/Tunnel Project along Interstate 93, total estimated cost $14.6 billion</td>
<td>Poor cost estimation, design and location analysis</td>
<td>ENR, May 3, 2004, p. 12</td>
</tr>
<tr>
<td>I-95/395/495 interchange in northern Virginia, initial cost estimates inflated by $435.5 million</td>
<td>Poor cost estimation, design and location analysis</td>
<td>ENR, <a href="http://www.enr.com/news/transportation">www.enr.com/news/transportation</a></td>
</tr>
</tbody>
</table>

The methods used for selecting routes and locations in those projects were based on engineering judgment, public/political input, and trial-error. Higher costs are associated with making redline (after the fact) changes to highway alignment deficiencies after construction. With that in mind, horizontal and vertical alignments should be designed in an automatic manner such that redline changes after the fact are not necessary. A number of costs are considered for highway construction by transportation agencies. A significant task in highway planning is to search for an economical route between given start and endpoints. This process requires precise
computations of highway costs [1-3, 9]. Although vast solutions may exist for an economical route, none may be feasible. Laborious trial and error methods don’t seem to provide a solution that is close to optimal. Issues concerning manual processes do not provide optimal solution to satisfying complex design measures.

This paper begins to discuss the very nature of highway alignments and integrating a GIS and genetic algorithm based optimized model into computer-aided design and drafting (CADD). Stemming from previous research conducted in highway alignment optimization and a model developed in earlier works for highway alignment optimization [2], we will attempt to integrate that model with an AutoCAD-based design model called, Road Designer 2000 (RD 2000). The integrated model will be used for enhancing the overall development and design of roadway alignments and displaying them via AutoCAD.

Through the utilization of CADD the model will be able to perform numerous calculations and conceptual designs based on the given data and to craft highway alignments promptly and efficiently. GIS and genetic algorithms implementation will provide us with digital terrain modeling of land mass features and more accurate searches for an optimal solution. Users will find it easy to plan, design, and make modifications accordingly.

2. Highway alignment optimization

Finding an optimal solution involves searching, calculating, and probing to reach the best possible extreme. The optimal solution is not always the best solution however, if all constraints are not incorporated effectively. Different circumstances may depend on different measures to be taken. Costs can be associated with applying the correct method of obtaining the perfect alignment. Highway alignment optimization (HAO) involves obtaining the alignment (horizontal and vertical simultaneously) by connecting two end-points (Figure 1) that best satisfies stated objectives and constraints.

Theoretically, the HAO problem can have an infinite number of alternatives to be evaluated. Optimizing highway alignments [2] requires capturing all sensitive and dominant costs, developing efficient solution algorithms, and working with real mapping and land mass information. When designing a new highway along a particular corridor, horizontal and vertical alignments pose distinct challenges. This can easily be seen since horizontal alignments particularly involve right-of-way contingencies [7] while vertical alignments prove a strong correlation to earthwork formulations. The qualifying comparison is seen when it comes to formatting the costs between the two. The RD 2000 model will be able to implement design data with genetic algorithms and GIS maps to construct alignments in CADD.

AASHTO [4] recommends that intersecting roads should generally meet at or near right angles. Kim et al. [5, 8] developed a local optimization procedure for intersections within an HAO model previously developed by Jong and Jha [2,3]. That procedure allowed acceptable intersection angles when designing intersections in accordance with the AASHTO [4] design criteria. Excessive curvature and poor combinations of curvature limit capacity, and cause economic losses because of increased travel time and operating costs. Alignments should: (1) be as direct as possible, (2) be aligned for the appropriate design speed, (3) be consistent with minimum curves, (4) avoid sharp curves, and (5) avoid reversals. Right-of-way and construction costs are considered to be the most significant factors contributing to financial woes in highway construction. Within the highway optimization model a search space is specified and depending on the total area covered, the model will gather several alignments to choose from. Environmental and historical areas must be considered beforehand.
Figure 1 assumes that the start and end points are given. This must be true to begin the process. The points of intersections ($P_i$) are assumed to fall along the orthogonal cutting lines (planes for the 3-dimensional case) passing through intermediate points placed at equally spaced intervals between the start and end points. The $P_i$'s are first connected with straight lines; curves are then fitted to connect straight lines (see, Figs. 2 and 3). The curve radius is calculated using the AASHTO [4] design criteria. Thus, the problem reduces to finding the $P_i$'s, which are treated as the decision variables for alignment optimization.

In Figure 3, $C_i$ and $T_i$ denote points of curvature and points of tangency, respectively. For notational convenience, we further denote $T_0 = P_0 = S$ and $C_{m+1} = P_{n+1} = E$ as the start and end points of the alignment. RD 2000 will integrate and generate composite cross-section views of the horizontal alignment of all elements pertaining to the design. The software will calculate station-by-station and will determine (vertically and horizontally) the location of intersecting points, elevation and their specified lengths contingent upon real time GIS data.

The program will generate 3-D perspective views for each station based upon specific locationing. Programming in “C” will be the backbone behind the meshing of underlying icons such as GIS and CADD.
3. Genetic algorithms and geographic information systems

Genetic algorithm (GA) is currently the most prominent and widely used optimization algorithm based on artificial intelligence, primarily due to its ability to search in a continuous space and avoid local optima. GA’s [3] have been proven to be effective in optimizing highway alignments due to their ability in optimizing horizontal and vertical alignments concurrently. They are particularly suited for exploiting the entire solution space while randomly looking for a better solution. Within each generation alignments are evaluated wherein the population of alignments are randomly created, which eventually lead to an optimal solution. The algorithms were successfully developed for optimizing 3-dimensional alignments. When these algorithms [2] were paired with GIS, the efficiency of the computations became important due large in part to the spatial modeling process and its ineptness for optimal searches. A GIS model can be directly applied to this situation if there is presently GIS data and mapping for the area in question.

Within the HAO model the algorithms will perform searches for the best alignments from station to station. Perspective view of the road will be the output. GIS data must accurately reflect the search space parameters, to accurately value the area and produce a quality result. Genetic algorithms were coded in “C” and integrated with ArcView GIS through dynamic link libraries [1].

4. Trade-off analysis

Opportunity costs are present in all endeavors whether it involves saving for a home or providing a ramp at an interchange with minimum radius to allow for a commercial building to be placed closer to the highway. These costs/trade-offs can be the difference in highway placement issues. The underlying trade-off factors that were discussed in earlier works concerning alignments are construction costs, life-cycle/maintenance costs, user delay, environmental sensitivity, demand of the region, noise and pollution, societal costs, public and political acceptability, accessibility to Central Business Districts (CBDs), security concerns, and interdependence with other transportation systems and modes. A criteria-based decision support system [6] was developed to perform a trade-off analysis that compared effects of costs, and environmental and right-of-way impacts among a set of highway alternatives. The system was designed to aid highway planners and designers in selecting the best highway by comparing similar qualities of a set of highway alternatives.
5. Design platform analysis

The focus will now turn to integrating the HAO model with the RD 2000 model and compatibility of the HAO model with other CADD-based softwares, such as MicroStation. One of the main thresholds in moving from one software package to the other is the terminology. Both RD2000 (an AutoCAD-based package) and MicroStation products have similar terminology as MicroStation uses levels and line styles while AutoCAD has layers and line types. Table 3 lists some of these similarities. Some of the differences between the two programs are even more distinct. AutoCAD considers the origin (0,0) to be in the lower-left of the drawing. MicroStation on-the-other-hand considers this origin point (0,0) to be in the center of the design plan. Also, the design plan in MicroStation is finite whereas in AutoCAD it is infinite space setting. Figure 4 illustrates the AutoCAD desktop platform as seen when opened initially with RD 2000. MicroStation has a built-in translator that allows for converting AutoCAD “DWG” files into the “DGN” format that it contains. Previous studies [2, 9] have shown that user travel-time and delay costs generally account for a higher percentage of total alignment cost. One must adhere to already specified design guidelines for constructing as well as improving a highway to an extent, trying to minimize these costs. Among the governing bodies that regulate these specifications are the American Association of State Highway and Transportation Officials (AASHTO) and Americans with Disabilities Act (ADA). Consequently, if the horizontal and vertical alignments are specified and terrain data are available, detailed design of a proposed 3-D highway section can be achieved. One circumstance that an automated design procedure might produce, with no guidance to the contrary, is the intersection of highways at an overly acute angle [5, 8]. As noted earlier, precision could play a major role when optimizing an alignment in preservation and environmentally sensitive areas. The design software RD 2000 runs on AutoCAD; but the alignment optimization can also be integrated to MicroStation to perform detailed design.

![Fig. 4 - A captured screen setting from RD 2000](image-url)
Tab. 3 - AutoCAD vs. MicroStation Terminology

<table>
<thead>
<tr>
<th>AutoCAD</th>
<th>MicroStation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layers = Levels</td>
<td></td>
</tr>
<tr>
<td>Blocks = Cells</td>
<td></td>
</tr>
<tr>
<td>Attributes = Tags</td>
<td></td>
</tr>
<tr>
<td>Polyline = Linestring</td>
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</tr>
<tr>
<td>Linetype = Linestyle</td>
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<td>Xref = Reference File</td>
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</table>

Tab. 4 - Types of output results from the HAO

<table>
<thead>
<tr>
<th>Type of output</th>
<th>Contents</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Costs breakdown for all searched alignments</td>
<td>Earthwork costs</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>Length dependent costs</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>Right-of-way costs</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>Penalty costs for gradient</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>Penalty for vertical curve</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>Structure cost (bridge and tunnel)</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>User costs</td>
<td>$</td>
</tr>
<tr>
<td></td>
<td>Alignment length</td>
<td>Feet</td>
</tr>
<tr>
<td>Earthwork cost (per station)</td>
<td>Elevation of alignments</td>
<td>Feet</td>
</tr>
<tr>
<td></td>
<td>Cut volume</td>
<td>Cubic yard</td>
</tr>
<tr>
<td></td>
<td>Fill volume</td>
<td>Cubic yard</td>
</tr>
<tr>
<td>Detail results for the optimized alignment</td>
<td>PI Index for horizontal and vertical alignment (X,Y coordinates and road elevation)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Number of horizontal and vertical curves</td>
<td>No.</td>
</tr>
<tr>
<td></td>
<td>Horizontal curve radius</td>
<td>Feet</td>
</tr>
<tr>
<td></td>
<td>Length of vertical curves</td>
<td>Feet</td>
</tr>
<tr>
<td>Coordinate of the optimized alignments (X, Y)</td>
<td>Residential relocations</td>
<td>X,Y coordinates</td>
</tr>
<tr>
<td>Environmental impact</td>
<td>Affected properties</td>
<td>No.</td>
</tr>
<tr>
<td>Summary</td>
<td>Areas affected by the optimized alignment</td>
<td>Square feet</td>
</tr>
</tbody>
</table>

6. Example study

The example of the proposed integration of HAO and RD2000 is best demonstrated by a real project on which our research team has recently worked. We have worked on the MD 97 Brookeville bypass project in the area of Brookeville, Maryland to demonstrate the practical applicability of the HAO model to find the best alignment connecting the given two endpoints.

Among the preferable optimized alignments the model generated, Figure 5 presents an optimized alignment with due environmental concerns and users’ specified design standards (e.g., 50 mph design speed, 40 feet of 2 lane road, 4 points of intersection, etc.).
Basically, the HAO can generate the information, which is essential in specific road designing, for all the evaluated alignments during the program run as the model output (see Table 4).

Figure 6 presents the flowchart of HAO/RD 2000 integration. The user specified input values (such as, design speed, road width, maximum super-elevation rate, etc.) and output results (such as information of point of intersections for horizontal and vertical alignments) of the optimized alignment are imported to RD 2000. These tasks can be easily conducted by introducing electronic forms of detail information of point of intersections for the horizontal and vertical alignment to RD 2000.

However, it is noted that since the HAO model is not automatically communicated with RD 2000 during the optimization process, the optimized results from the HAO are transmitted to RD 2000 after model termination.

Figures 7 and 8 show the horizontal and vertical shapes of the optimized alignment within the RD 2000 platform.

Figure 9 presents a 3-dimensional view of the optimized alignment at a certain station point. Basically, RD 2000 automatically calculates the stopping and passing sight distances from any specified station by taking into account the 3-dimensional road geometry, and allows users to instantly determine whether the sight distance will be obstructed by supplemental structures. Thus, we expect that it is possible to generate a 3-dimensional view of the optimized alignment at any station with the HAO/RD 2000 integrated model.
Fig. 5 - An optimized alignment from the HAO model

Fig. 6 - HAO/RD 2000 integration flow-chart
Fig. 7 - Horizontal shape of an optimized alignment in RD 2000

Fig. 8 - Vertical shape of an optimized alignment
7. Conclusion

The contribution in this paper is expected to be pivotal in automated road design and optimization. The overall objective of this work is aimed at automating the road design and alignment optimization procedure by minimizing the total cost of alignments, construction, and reconstruction of transportation facilities (namely highways) by implementing a once manual procedure into a systematic software-based procedure. The paper also addresses environmental issues and maintenance to transportation facilities during road design and alignment selection.

The overall benefit of the integrated HAO-RD 2000 model will contribute to a more accurately designed highway. Integration of the HAO model with multiple design platforms will offer a means of user choice and perception. Future works will strive to generate improved methods of interfacing intersecting roads through albeit intersections or interchanges, of fitting roads segments into larger networks, of scheduling improvements in road networks, and of displaying future road characteristics through visualization techniques to ascend where existing commercial products immobilized. With the advent of MicroStation as a perspective platform the user interface will be expanded. Additional factors, such as noise and noise wall modeling, traffic control, other roadside design features, and reconstruction can also be considered in the HAO model in future works.

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References

Assessment of unprotected left-turn capacity models using arena simulation

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Abstract
Gap acceptance theory based on the stochastic process is commonly used to analyze unprotected left-turn capacity at two-way stop-controlled intersections or signalized intersections with a permitted left-turn phase. However, there are some factors in the current HCM models that are found to confound the stochastic process. This paper presents new models to improve capacity estimation for the unprotected left turn. According to the simulation results using the Arena models, it was found that for TWSC intersections, if there are a relatively higher number of heavy vehicles in the left-turn traffic, the HCM model may underestimate the left-turn capacity. For signalized intersections, the HCM model tends to under-estimate the unprotected left-turn capacity for the lower opposing-through traffic volume and over-estimate it for the higher volume. The heavy vehicles in the left-turn traffic can increase the under-estimation trend. Furthermore, the simulation results verified that the proposed new models are reliable.

Keywords – TWSC intersections, signalized intersections, gap acceptance, traffic capacity, unprotected left-turn, and Arena simulation

1. Introduction

For the unprotected left-turn traffic at two-way stop controlled (TWSC) intersections or during the permitted left-turn green phase at signalized intersections, capacity estimations are based on the gap acceptance theory. In the current HCM capacity estimation models (HCM, 2000), besides the conflicting volume, the critical gap and follow-up time are key factors to determine the potential unprotected left-turn capacity. As a constant value, the critical gap is the minimum gap that all drivers in the minor stream are assumed to accept under the similar intersection geometric conditions. Usually, the left-turn vehicles (those yielding right of way) enter in the long gaps at headways often referred to as the follow-up time. Although it is not realistic to assume that all left-turn drivers accept a constant critical gap and use a consistent follow-up time, the guides of updating HCM (1985, 1994, 1997, and 2000) kept being based on this assumption. The chapter
for unsignalized intersections in Traffic Flow Theory indicated that the assumption of driver’s consistent and homogeneous gap-acceptance behavior is valid and the overall effect of the assumption on determining the left-turn capacity is minimal (Troutbeck and Brilon, 1997). However, there are still several important procedures in the current HCM capacity models for unprotected left turns that are found to confound the stochastic process of gap acceptance. For example, at TWSC intersections, if there is a certain percent of heavy vehicles among the left-turn traffic, a larger critical gap and follow-up time are needed by the heavy vehicles compared to passenger cars. Considering the effect of heavy vehicle factor on the traffic capacity, the HCM simply increases the critical gap and follow-up time by an adjustment, which is equal to the extra time needed by heavy vehicles multiplying the proportion of the heavy vehicles. Obviously, this method is not realistic and may cause errors for the capacity estimation.

For signalized intersections with a permitted left-turn phase, the unprotected left-turn capacity estimation in the HCM is based on saturation flow rate estimation for critical lane groups. Through adjusting the left-turn factor, it simply combines the unprotected left-turn group with other groups such as going through, right turn, and protected left turn. The signalized intersection chapter in the HCM does not consider the effect of the number of lanes in the opposing-through movement on the unprotected left-turn traffic. However, in the HCM chapter for unsignalized intersections and other previous literatures (AASHTO, 2001; Kyte et al., 1996; Chang et al., 1996), it is clearly pointed out that the more opposing through lanes there are, the larger critical gaps and follow-up times are needed by left turners. Furthermore, after the beginning of the green phase at a signalized intersection, the unprotected left-turn vehicle should have to wait at the stop line for a proper gap in the opposing traffic. Such gap is not possible in general until the queue of the opposing through movement accumulated during the red phase is cleared. The portion of effective green blocked by the clearance of an opposing queue of vehicles significantly influences the left-turn capacity. The estimation method for the portion of effective green is based on regression relationship with other factors such as signal phase and opposing-through traffic, which may also have a negative effect on the stochastic process. Therefore, the aforementioned issues may accumulatively result in estimation errors for the permitted left-turn capacity at signalized intersections.

To improve the capacity estimation procedures, the objectives of this study are put forward as to:
- evaluate the current HCM capacity models for the unprotected left-turn scenarios at TWSC intersections and signalized intersections using a simulation software (ARENA),
- adjust the capacity models to enhance the accuracy of capacity estimation, and
- validate the adjusted models based on the Arena simulation.

2. Left-Turn Capacity Estimation for TWSC Intersections

2.1 The current HCM Capacity Model

In the HCM gap-acceptance methodology, the priority of right-of-way given to each traffic stream must be identified. Some streams have absolute priority, whereas others have to give way or yield to higher-order streams. Figure 1 shows the relative priority of streams at four-leg intersections. The conflicting movements for the left-turn movement (e.g., movement 1) include opposing-through, right-turn, and pedestrian movements (e.g., movements 5, 6, and 16). The potential traffic capacity for the unprotected left-turn movement is dependent on the total conflict flows, as shown in Figure 1-b. In the HCM, the method does not differentiate between crossing and merging conflicts. Left turns from the major street and the opposing right turns from the
major street are considered to merge, regardless of the number of lanes provided in the exit roadway. Pedestrian flow rates are also included as a part of the conflicting flow rates, since they, like vehicular flows, define the beginning or ending of a gap that may be used by a minor-stream vehicle.

The gap acceptance model used in HCM computes the potential capacity of each minor traffic stream in accordance with Equation 1.

\[
C_{p,x} = \frac{V_{c,x} \exp(-V_{c,x}t_{c,x}/3600)}{1 - \exp(-V_{c,x}t_{f,x}/3600)}
\]  

(1)

where,

- \(C_{p,x}\) = potential capacity of minor movement \(x\) [veh/h],
- \(V_{c,x}\) = conflicting flow rate for movement \(x\) [veh/h],
- \(t_{c,x}\) = critical gap for movement \(x\) [s], and
- \(t_{f,x}\) = follow-up time for minor movement \(x\) [s].

According to the Highway Capacity Manual (2000), the critical gap \((t_{c,x})\) is computed separately for each minor movement by Equation 2. Adjustments are made for the presence of heavy vehicles.

\[
t_{c,x} = t_{c,base} + t_{c,HV}P_{HV}
\]  

(2)

where,

- \(t_{c,base}\) = base critical gap [s],
- \(t_{c,HV}\) = adjustment factor for heavy vehicles [s],
- \(P_{HV}\) = proportion of heavy vehicles for minor movement.

a. Priority of traffic streams at an intersection       b. Conflicting flows with unprotected left turn

Fig. 1 – Priority of traffic streams at an intersection and conflicting flows
The follow-up time \( t_{f,x} \) is computed for each minor movement using Equation 3.

\[
t_{f,x} = t_{f,\text{base}} + t_{f,HV} P_{HV}
\]  

(3)

where,

\[ t_{f,\text{base}} \] = base follow-up time [s]

\[ t_{f,HV} \] = adjustment factor for heavy vehicles, and

If there is a certain percent of heavy vehicles among the minor traffic, a larger critical gap and follow-up time are needed by the heavy vehicles compared to the passenger cars. For example, the AASHTO manual (2001) states that for left turns from the major road, the critical gap is 5.5 s for a passenger car, 6.5 s for a single-unit truck, and 7.5 s for a combination trucks. According to the current HCM (2000), for a passenger car left-turn maneuver, the base critical gap is 4.1 s, which is assumed to be independent of the number of lanes for a major road street with less than 6 lanes, and the basic follow-up time is 2.2 s. For heavy vehicles, the critical gap is 1.0 s more for two-lane major streets and 2.0 s more for four-lane major streets, and the follow-up time is 0.9 s more for two-lane major streets and 1.0 s more for four-lane major streets, compared to passenger cars. The HCM model just simply adjusts critical gaps and follow-up times between heavy vehicles and passenger cars as constant values by the heavy vehicle proportion.

2.2 Proposed model to estimate for minor movement at TWSC intersections with high percent heavy vehicles

To more accurately estimate capacity for minor movement at TWSC intersections with high percent heavy vehicles, a proposed capacity model is proposed as Equation 4.

\[
C_{p,x} = V_{c,x}(1 - P_{HV}) \frac{\exp(-V_{c,x}t_{PC,c,x}^{PC} / 3600)}{1 - \exp(-V_{c,x}t_{PC,f,x}^{PC} / 3600)} + V_{c,x} P_{HV} \frac{\exp(-V_{c,x}t_{HV,c,x}^{HV} / 3600)}{1 - \exp(-V_{c,x}t_{HV,f,x}^{HV} / 3600)}
\]  

(4)

where,

\[ V_{c,x} \] = conflicting flow rate for movement \( x \) [veh/h],

\[ t_{PC,c,x} \] = critical gap of passenger cars for minor movement \( x \) [s],

\[ t_{PC,f,x} \] = follow-up time of passenger cars for minor movement \( x \) [s],

\[ t_{HV,c,x} \] = critical gap of heavy vehicles for minor movement \( x \) [s], and

\[ t_{HV,f,x} \] = follow-up time of heavy vehicles for minor movement \( x \) [s]

\[ P_{HV} \] = proportion of heavy vehicles for minor movement.

Since gap acceptance behavior for heavy vehicles is significantly different from passenger cars, the proposed model separates their capacity estimations and then integrates them based on their discrete probability distribution (proportion of heavy vehicles).

2.3 Basic Arena model without heavy vehicles in the left-turn traffic

The Arena Simulation Model (Kelton et al., 2004) developed by Rockwell Software Inc. is used to simulate a stochastic process that unprotected left-turn traffic to cross opposing-through
traffic on a four-lane major road to enter the minor road. The Arena software package is mainly used for creating animated models and representing any system virtually. Arena supports all types of applications and has about 60 inbuilt modules. It is a very useful tool for simulation of transportation systems, telecommunication queuing systems, and manufacturing materials processes (Nagarajan et al., 2002). The Arena simulation for the left-turn gap acceptance in this paper is similar to a previous Arena application for pedestrian gap acceptance at roundabout areas (Wan and Rouphail, 2004). In this previous study, the model was built to simulate a composite queuing system of pedestrian crossings at roundabout areas.

To evaluate the capacity model, a basic Arena model is first designed to simulate the stream interactions between major road conflicting opposing-through traffic and unprotected left-turn traffic without heavy vehicles. According to the HCM, the process of the left-turn gap acceptance is an M/M/1 arrival-queuing-departure system with only one left-turn lane. It assumes that vehicles in both streams arrival at random to conform to a Poisson distribution with a constant rate. Therefore, the inter-arrival times between vehicles follow an exponential distribution, which is shown as the following formulation to generate traffics:

\[ i_1 \sim \exp(\beta_1) \quad \text{for the opposing-through traffic} \]
\[ i_2 \sim \exp(\beta_2) \quad \text{for the unprotected left-turn traffic} \]

Where \( i_1 \) and \( i_2 \) are inter-arrival times of vehicles (gap), and \( \beta_1 \) and \( \beta_2 \) are distribution parameters equivalent to the reciprocals of vehicle demand.

For the opposing-through traffic, whenever a vehicle is generated, its inter-arrival time \( i_1 \) to the last vehicle is recorded and assigned as a gap to be compared to the constant critical gap in a decision module. Without heavy vehicles in the left-turn traffic, the critical gap is 4.1 s and the follow-up time is 2.2 s for passenger cars. If the current gap is smaller than the critical gap, the left-turn vehicle rejects it and waits for the next acceptable one. If the current gap is larger than the critical gap, the opposing through vehicle sends a signal to allow the left-turn vehicle to use it and cross the intersection. When the larger gap is accepted, the gap should lose 2.2 s following-up time as a lag. Then, the lag will go back to the decision module to be compared to the critical gap again. If the lag is still larger, it will send the signal again and then be subtracted by 2.2 s.

This cycle logic continues until the lag is useless and smaller than the constant value of 4.1 s. Summarizing the above simulation process, Figure 2-a illustrates the Arena flowchart for the opposing-through traffic. For the left-turn traffic, when a vehicle is generated, it first enters a hold module that stand for a left turn lane to wait for the signal (the larger gap or lag). If the current gap is small, it will stay at the left-turn lane until a signal is available and the continuously coming left-turn vehicles will accumulate queue there. The rule for the queuing is equivalently First-In-First-Out (FIFO) in the M/M/1 system. Figure 2-b illustrates the Arena flowchart for the left-turn traffic. Fifty-replication simulations were conducted in the basic scenario for each opposing-through volume individually increased by 200 veh/h. For each replication, 5 minutes warm-up period was used to put them into a steady state and then one hour effective simulation time was performed to collect data. To obtain the potential left-turn capacity, the left-turn volume should be higher than the left-turn capacity and therefore was set by 3600 veh/h \( (\beta_2=2 \text{ s}) \). Thus, multiple waiting queues have to be formed and the number of vehicles departing from the left-turn lane is the simulated capacity. The average capacity simulation results and their 95% confidence intervals based on the basic Arena model are shown in Table 1. It is not surprising that the simulation results exactly fit the calculated results from the HCM Equation 1, because the simulation model was based on the HCM assumption for passenger cars. The equivalent results also verified that the basic model logic is correct.
Fig. 2 – Basic Arena model to simulate unprotected left-turn process at TWSC intersections

### Tab. 1 – Comparison between calculation results of the HCM model and simulation results of the basic scenario without heavy vehicles

<table>
<thead>
<tr>
<th>Conflicting volume (veh/hour)</th>
<th>HCM results (veh/hour)</th>
<th>Mean (veh/hour)</th>
<th>95 % C.I. (veh/hour)</th>
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<tbody>
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<td>1800</td>
<td>347.3</td>
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<td>200</td>
<td>1384.3</td>
<td>1389.3</td>
<td>1383.6</td>
</tr>
</tbody>
</table>

### 2.4 Test Arena model with heavy vehicles in the left-turn traffic

Since the passenger cars and heavy vehicles need different critical gaps and follow-up times, a decision module was designed to separate requirements of the critical gap and follow-up time for passenger cars and heavy vehicles. If there is a passenger car waiting for left turn, the critical gap and follow-up time keep the same as the basic scenario; if there is a heavy vehicle, the critical gap is 6 s and the follow-up time is 3.2 s based on the HCM (2001). Figure 3-a illustrates the Arena flowchart for the opposing-through traffic. For the unprotected left-turn traffic, the proportion of the heavy vehicles ($P_{HV}$) can be assigned by discrete accumulative probability (DISC) to separate heavy vehicles and passenger cars. Figure 3-b illustrates the Arena flowchart for the left-turn traffic. It can be formulated as the following expression in Arena:

$$\text{DISC}(P_{HV}, 1, 1.0, 2)$$

Where, $P_{HV}$ is the probability for heavy vehicles in the left-turn traffic, the first ‘1’ stands for the heavy vehicle, the second ‘1.0’ is the accumulative probability, and the ‘2’ stands for the passenger cars.
Based on Equations 1, 2, and 3 from the current HCM capacity model, with a certain percent heavy vehicles in the left-turn traffic, the following expression should be used for the 4-lane highway:

\[
C_{p,x} = V_{c,x} \frac{\exp[-V_{c,x}(4.1 + 2P_{HV})/3600]}{1-\exp[-V_{c,x}(2.2 + P_{HV})/3600]}
\]

The calculation results from the above formula are listed in Table 2 with the results from the proposed model together. The table also shows the average capacity simulation results and their half widths of 95% confidence intervals based on the test models with 0.1 and 0.2 percent heavy vehicles (which simulation running settings are as same as the basic scenario). It is found that the current HCM model always under-estimates the left-turn traffic capacity with high percent heavy vehicles. However, there is no significant difference between simulation results and calculation results based on the new model. The test Arena simulation verified the results achieved is reliable.
Tab. 2 – Comparison between the HCM model, the proposed model, and simulation results of the test scenario with heavy vehicles

<table>
<thead>
<tr>
<th>Vc</th>
<th>HCM results (veh/h)</th>
<th>Prop. results (veh/h)</th>
<th>Simulation Results (veh/h)</th>
<th>HCM results (veh/h)</th>
<th>Prop. results (veh/h)</th>
<th>Simulation Results (veh/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1800</td>
<td>306.8</td>
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<td>387.4</td>
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<td>456.2</td>
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<tr>
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</table>

Fig. 4 – Capacity estimation deviations from the current HCM model compared to the new model

Figure 4 illustrates left-turn capacity deviations from the current HCM model compared to the new model for different proportions of heavy vehicles from 0.1 to 0.5 percent. It shows that the deviation caused by the current model increases as the conflicting volume increase. For 1800 veh/h conflicting traffic, the error rate could be at least 5%. It was also found that the error rate increases with the increment of the heavy vehicle proportion. For a 50% of heavy vehicles, the error rate can be more than 16% for 1800 veh/h conflicting traffic. Although that very high proportion of heavy vehicles is very unusual in most areas, it is interesting to observe that if we keep enlarging the heavy vehicle proportion, the error increasing trend will stop at \( P_{HV} = 0.6 \), and then will gradually decrease at zero till \( P_{HV} = 1.0 \).

3. Unprotected Left-Turn Capacity Estimation for Signalized Intersections

3.1 The current HCM Capacity Model

In the HCM chapter for signalized intersections, capacity estimation is based on the concept of saturation flow rate. The saturation flow rate is the flow in vehicles per hour that can be accommodated by the lane group assuming that the green phase was displayed 100 percent of the
time (i.e., \( g/C = 1.0 \)). A saturation flow rate for the unprotected left-turn group is computed according to Equation 5. The unprotected left-turn capacity may be calculated by Equation 6.

\[
s = s_0 f_{HV} f_{LT}
\]

\[
C_L = s \frac{g}{c}
\]

where

- \( s \) = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group [veh/h],
- \( s_0 \) = base saturation flow rate per lane [1900 pc/h/ln],
- \( f_{HV} \) = adjustment factor for heavy vehicles in traffic stream,
- \( f_{LT} \) = adjustment factor for left turns in lane group,
- \( C_L \) = left-turn capacity of an exclusive permitted left-turn group [veh/h],
- \( g \) = effective green time for subject permitted left turn [s], and
- \( c \) = the length of the cycle [s].

The effect of heavy vehicles is treated by adjustment factor \( f_{HV} \), which is calculated by Equation 7. The heavy-vehicle factor accounts for the additional space occupied by these vehicles. The passenger-car equivalent (\( E_T = 2 \)) is used for heavy vehicles, which means each one heavy vehicle can be considered as 2.0 passenger-car units.

\[
f_{HV} = \frac{100}{100 + \%HV (E_T - 1)}
\]

The left-turn adjustment factor \( f_{LT} \) is summed as Equation 8 for the exclusive lane with permitted left-turn phasing.

\[
f_{LT} = (g - g_q) / g E_{LT}
\]

where

- \( g \) = effective green time for unprotected left turn [s],
- \( g_q \) = the portion of effective green blocked by the clearance of an opposing queue of vehicles [s], and
- \( E_{LT} \) = through-car equivalents.

When the green is initiated, the opposing queue begins to move. While the opposing queue clears, left turns from the subject direction are effectively blocked. The portion of effective green blocked by the clearance of an opposing queue of vehicles is designated as \( g_q \). If assuming that there are no effects of lane utilization factor and platoon ration for opposing traffic, \( g_q \) can be calculated by Equation 9, which is based on regression relationship with other factors.

\[
g_q = \frac{v_{oc} (1 - g/c)}{0.5 - v_{oc} (1 - g/c)} - t_L
\]

where

- \( v_{oc} \) = adjusted opposing flow rate per lane per cycle,
- \( g \) = effective green time for subject permitted left turn [s],
- \( c \) = the length of the cycle [s], and
- \( t_L \) = lost time for opposing lane group [s].
Furthermore, the term through-car equivalent \((E_{LT})\) is calculated by Equation 10.

\[
E_{LT} = \frac{s_{HT}}{s_{LT}}
\]

Where

\(s_{HT} = \text{basic saturation flow of through traffic (veh/h/ln)} = 1900 \text{ veh/h/ln}, \) and

\(s_{LT} = \text{filter saturation flow of unprotected left turns (veh/h/ln), which is computed by Equation 11.}
\]

\[
s_{LT} = \frac{V_o \exp(-V_o t_c / 3600)}{1 - \exp(-V_o t_f / 3600)}
\]

Where

\(V_o = \text{opposing-through traffic flow rate,}\)

\(t_c = \text{critical gap (4.5 s), and}\)

\(t_f = \text{follow-up headway (2.5 s).}\)

3.2 Proposed Model based on stochastic process

The HCM uses larger critical gap and follow-up time for signalized intersections than those for TWSC intersections and it did not consider effect of the number of lanes in the opposing-though movement on the permitted left-turn traffic. However, there should be no significant difference in the left-turn gap-acceptance behaviors between signalized intersections and TWSC intersections under the similar traffic conditions and geometric features. Therefore, a proposed capacity model for the permitted left-turn traffic based on the stochastic process is developed as Equation 12. In the formula, filter saturation flow estimation of unprotected left turns \((s_{LT})\) is consistent with the capacity calculation for unprotected left-turn at TWSC intersections \((C_{p,x})\) from Equation 4.

\[
C_{p,x} = \frac{S_{LT}}{P_t} = C_{p,x} P_t
\]

Where

\(C_{p,x} = \text{capacity for permitted left-turn movement x at signalized intersections (veh/h)},\)

\(P_t = \text{the proportion of the effective green time that can be used for left-turn operation to the whole cycle length, which can be calculated by Equation 13.}\)

\[
P_t = \frac{g - g_q - t_c}{c}
\]

Where, the portion of effective green blocked by the clearance of an opposing queue of vehicles \((g_q)\) can be calculated based on the HCM queue accumulated polygon (QAP) representing the total stopped time of vehicles arriving per cycle. Figure 5 presents a time-space diagram illustrating a triangle shaped polygon representing the opposing-through traffic flow during a cycle. It shows that vehicles uniformly arrive at and leave a stop line at the vehicle arrival rate, \(q_c\) [veh/s], and at the saturation flow rate, \(s_i\) [veh/s], respectively. Vehicles accumulate in queue at a stop line during the red phase \((r)\) and begin to depart during the green phase \((g)\) after the lost time \((t_l)\). Based on the time-space diagram, the term \(g_q\) can be computed by Equation 14.

\[
g_q = \frac{(r + t_c) q_i}{s_i - q_i}
\]
where
\[ q_t = \text{average arrival rate of opposing-through vehicles [veh/sec/ln]}, \]
\[ r = \text{length of red phase [sec]}, \]
\[ s_t = \text{saturation flow rate of opposing-through vehicles [0.528 veh/sec/ln]}, \]
\[ t_L = \text{lost time} \]

Compared to the HCM model, the calculation results of \( g_q \) based on the proposed model are larger for the lower opposing traffic volume but smaller for the higher traffic volume, as shown in Table 3. For the 200 veh/h opposing traffic, the \( g_q \) from the HCM model is even negative. It is unrealistic and the error may be resulted from the regression methodology.

### 3.3 Arena simulation model for the permitted left-turn at signalized intersections

An Arena model for the unprotected left-turn process at an isolated signalized intersection was developed, supposing that the left-turn traffic at an exclusive left-turn lane is crossing the two-lane opposing through traffic. Figure 6 shows the simulation model structure for the opposing through traffic under a fixed signal phase with 30 s red and 60 s green. The opposing-through vehicles in each lane are assumed to arrive at random to conform to a Poisson distribution with equivalent lane utilization. After the vehicles are created, the gaps among the traffic are recorded and the vehicles enter the respective lanes by a decision module.

![Diagram](image)

**Fig. 5 – The HCM defined queue accumulated polygon representing the opposing-through traffic**

**Tab. 3 – Comparison of the term \( g_q \) between the HCM model and proposed model**

<table>
<thead>
<tr>
<th>Vc</th>
<th>( q_t ) (Veh/h)</th>
<th>( r ) (Sec)</th>
<th>( g )</th>
<th>( t_L ) (Sec)</th>
<th>( g_q ) from HCM Model (Sec)</th>
<th>( g_q ) from Prop. Model (Sec)</th>
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<tr>
<td>2400</td>
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</tr>
<tr>
<td>200</td>
<td>2.5</td>
<td>30</td>
<td>60</td>
<td>2</td>
<td>0.2</td>
<td>3.8</td>
</tr>
</tbody>
</table>

- 63 -
The capacity of the two opposing-through lanes is controlled by a time schedule module: it is equal to zero during the red period and lost time (2 s) and the coming vehicles are accumulated into queue; it is equal to one during the green period (vehicles in the queue depart from the lane resource one by one with 1.895 s delay, which is equivalent to 1900 veh/h/ln saturation departure rate). After the opposing queues in both lane resources are cleared, the gaps among the opposing-through vehicles arriving during the effective green are possible to be used by the unprotected left-turn vehicles. Those vehicles with larger gaps compared to critical gaps that can be accepted will send signals to allow left-turn heavy vehicles or cars to cross the intersection. This procedure is exactly the same as that in the previous Arena model for TWSC intersections, as shown in Figure 3. Using the Arena model, fifty-replication simulations were conducted for each opposing-through volume individually increased by 200 veh/hour. The simulation results (with $P_{HV}=0$, $P_{HV}=0.1$, and $P_{HV}=0.2$) were used to compare with the capacity calculation results from the HCM model and proposed model, as illustrated in Figures 7-a, 7-b, and 7-c. It shows that the simulation results fit with calculation results from the proposed model very well, which verified that the proposed model is reliable. However, the HCM model under-estimates the left-turn capacity for the lower opposing-through traffic volume and over-estimates it for the higher volume. Moreover, there is a trend that the under-estimation rates are increasing as the percentages of heavy vehicles increase. The HCM capacity model just simply treated each heavy vehicle as 2 passenger-car unit equivalent, but did not consider the gap acceptance difference between heavy vehicles and passenger cars, which enlarges the effect of heavy vehicles on the capacity.

![Arena simulation model structure for the opposing-through traffic and left-turn traffic at signalized intersection with permitted left-turn phase](image-url)
Fig. 7 – Comparison of permitted left-turn capacity at signalized intersections between the HCM model, the proposed model, and Arena simulation results

4. Conclusion

Through a simulation study to describe the left-turn gap-acceptance processes at TWSC intersections and at signalized intersections with a permitted left-turn phase, this paper evaluated the current HCM capacity models. It was found that for TWSC intersections, if there is a certain
percent of heavy vehicles in the left-turn traffic, the HCM model may under-estimate the left-turn traffic capacity. The current HCM methodology that compromises of critical gaps and follow-up times between heavy vehicles and passenger cars as new constant values may confound the stochastic process and consequently affect the capacity calculations. For unprotected left turns at signalized intersections, the HCM model tends to under-estimate the left-turn capacity for the lower opposing-through traffic volume and over-estimate it for the higher volume and the heavy vehicle effect can increase the under-estimation trend. In the HCM method, there are several key calculation steps that may confound the gap acceptance process and cause capacity estimation errors. They include determining saturation flow rates, adjusting the heavy vehicle factor, and calculating the blockage time of the opposing-through queue during the effective green time. Proposed models were developed based on the stochastic process to improve the capacity estimation for unprotected left-turn traffic at both TWSC intersections and signalized intersections. The models calculation results were verified by the Arena simulation models.

To address the current issues existing in the HCM, the paper mainly focused on the gap-acceptance methodology of unprotected left-turn capacity estimation for the basic traffic condition. It did not extend the model application to more complex situation such as shared-lane capacity, pedestrian impedance, upstream signalized intersection effect, and platoon event in the opposing-through traffic. More research efforts are needed to explore the model applications on more realistic traffic situation. Furthermore, the proposed models were only verified by simulation results. So, a field study is still suggested to validate the models.

References
Estimation of seat belt effectiveness values using double pair comparison method based on state highway crash data

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Abstract
The purpose of this study was to estimate the seat belt effectiveness in preventing fatal and nonfatal injuries to motor vehicle occupants when they are involved in crashes. The double pair comparison method was used and the estimations were based on police reported highway crash data of the state of Kansas in the United States. Two vehicle groups were considered: passenger cars and other passenger vehicles (vans and pickup trucks). Only front seat occupants who are older than 14 years of age were considered in the analysis. Based on estimations, seat belts were found to be 53% effective in reducing fatal injuries to front seat occupants in passenger cars. In other passenger vehicles, effectiveness of seat belts in reducing fatal injuries is 57%. As far as nonfatal injuries are concerned, seat belts are 52% and 42% effective in reducing incapacitating and non-incapacitating injuries respectively in passenger cars. It was also found that seat belts are 34% effective in reducing possible injuries to front seat occupants in passenger cars. Seat belts reduce incapacitating injury risk to occupants in other passenger vehicles by 47%, while they reduce non-incapacitating injury risk by 42% in the same vehicle group. In addition, seat belts are 28% effective in reducing possible injuries to occupants in other passenger vehicles.

Keywords – Seat belt effectiveness, double pair comparison, KABCO, injury severity, motor vehicle occupants

1. Introduction
Restraint systems in motor vehicles have been found to be very effective in reducing injuries to motorists when they are involved in crashes. In general, seat belt effectiveness is defined as the percentage reduction in injuries to restrained occupants compared to those of unrestrained occupants. According to the National Highway Traffic Safety Administration (NHTSA) estimations based on national data in the United States, seat belts are about 45% effective in preventing fatal injuries and 60% effective in preventing nonfatal injuries [1].

Because of higher safety benefits associated with seat belt usage, as of 2006, all states in the USA, except New Hampshire, have enacted seat belt laws to mandate the use of seat belts. Out of those states, primary seat belt law is enforced in 25 states while rest of the states enforces secondary law [2]. Primary law allows the police officers to stop and cite the motorists for not wearing the seat belt. However, in the case of secondary law, motorists not wearing the seat belts can be cited only if they are stopped for some other traffic related violation.
Since the NHTSA seat belt effectiveness values have been estimated based on national data and thus represent average conditions, use of those effectiveness values in state level safety analyses may not be accurate in some cases. For example, rural highways account for 91% of total highway mileage in the state of Kansas, and about 50% of vehicle miles occur on those highways, compared to only 16% of vehicle miles of travel on all US rural highways [3]. Therefore, the seat belt usage patterns in Kansas may be different from what is observed nationally. In addition, since Kansas enforces secondary seat belt law, the seat belt usage trends in Kansas may be different from what is observed in states with primary seat belt laws.

The seat belt effectiveness values estimated by NHTSA are based on Abbreviated Injury Scale (AIS). AIS is a method used to rank the injury severity based on a scale ranging from 1 (minor injuries) to 6 (fatal injuries). KABCO is another scale used to measure the injury severity associated with motor vehicle crashes, which has 5 different levels to rank the injury severity (K-fatal, A-incapacitating, B-non-incapacitating, C-possible, and O-no injury). Almost all states in the USA use KABCO injury severity scale in their highway crash databases to report injury severities. Because of this incompatibility in injury severity scales, it is somewhat difficult to combine information from these two sources when conducting safety analyses. For example, to assess the effectiveness of a local safety belt promotion program in terms of number of injuries prevented due to increased seat belt usage, the KABCO injuries have to be converted into AIS injuries, if the state’s crash data base uses KABCO scale. For this purpose, conversion factors are needed and in most of the cases, such conversion factors are not available at state levels.

The objective of this study was to estimate the effectiveness of seat belts in reducing injuries to motor vehicle occupants based on KABCO injury severity scale. The double pair comparison method was used to estimate the seat belt effectiveness using police reported crash data in Kansas.

2. Literature Review

The NHTSA estimated the effectiveness of restraint systems in reducing fatalities and injuries in 1984 in its Final Regulatory Impact Analysis [4]. Both manual and automatic seat belts were considered. The study was based on data from National Crash Severity Study (NCSS) and National Accident Sampling System (NASS) for the period from 1979 to 1982. The estimation method was based on rate of restrained and unrestrained passengers who were injured in crashes. The results showed that the effectiveness of lap/shoulder belts in preventing fatalities was 40 to 50%, and the effectiveness of reducing nonfatal injuries was 45 to 55%. When lap/shoulder belts were combined with air bags, the estimated effectiveness was 45 to 55% for fatalities and 50 to 60% for nonfatal injuries. However, one of the shortcomings of this method was the difficulty in controlling the seat belt effectiveness for effects of other factors due to the insufficient availability of crash data during that period of time.

The above seat belt effectiveness values were later evaluated by NHTSA in a series of studies using more recent crash data [1,5,6]. These studies were conducted using data from the Fatality Analysis Reporting System (FARS) and NASS Crash Worthiness System (CDS). According to the latest study of this series, when considering seat belts alone (manual-shoulder belts), the effectiveness of seat belts in preventing fatalities was 45%. In addition, seat belts were found to be 60% effective in preventing non-fatal injuries (moderate to greater injuries) [1]. Both these values are similar to NHTSA’s original estimations from its 1984 study [4].

The double pair comparison method introduced by Evans [7] has been widely used by many researchers to estimate the seat belt effectiveness. Using this method, Evans [8] estimated the seat belt effectiveness in preventing fatal injuries based on crash data from Fatality Accident.
Reporting System (FARS) for the period from 1975 to 1983. It was found that the overall seat belt effectiveness in preventing fatal injuries to front seat passengers in passenger cars was 41% with an estimated error of 3%.

In order to estimate the effectiveness of rear seat restraint systems in preventing fatalities, Evans [9] analyzed FARS data from 1975 to 1985 using double pair comparison method. The subject occupant was considered as the right or left rear seat passenger. The subject occupant was compared with other occupants such as driver, right front passenger, and right or left rear passenger depending on the subject occupant being considered. The estimations showed that the seat belts are 9% to 27% effective in preventing fatalities to rear seat passengers.

Kahane [10] also used double pair comparison method to estimate seat belt effectiveness in preventing fatalities in passenger cars and light trucks using FARS data for two time periods: 1977 to 1985 and 1986 to 1999. The main objective was to examine the appropriateness of NHTSA’s long-standing estimates of seat belt effectiveness values, which were based on FARS data before 1986, for more recent FARS data. An empirical tool was developed to adjust for the biases in double pair analyses of later FARS data. Results reconfirmed the NHTSA’s earlier seat belt effectiveness estimates of 45% in passenger cars and 60% in light trucks against fatalities.

Kahane [11] estimated the fatality and injury reducing effectiveness of lap belts for rear seat occupants using double pair comparison method. The subject occupant was considered as the backseat passenger while the other occupant was the driver. The seat belt effectiveness was estimated for both fatal and nonfatal injuries. The injury severity levels were classified as, serious (categories K and A in KABCO scale), moderate to serious (categories K, A, and B), and overall injury severity (all categories). Crash data from two data sources were used: FARS data from 1975 to 1976 and crash data from Pennsylvania for 1982 to 1985. The estimations showed that lap belts are 17% to 26% effective in preventing fatal injuries to back seat occupants. In addition, lap belts were found to be 37% effective against serious injuries, while they are 33% effective against moderate to serious injuries.

Cummings et al. [12] used Conditional Poisson regression method to estimate seat belt effectiveness in preventing fatal injuries to motor vehicle occupants. In this method, relative risk between two occupants was estimated using matched-pair cohort data while controlling for variables such as occupant gender, seating position, and age. Using FARS data from 1975 to 1998, they estimated that the risk of death for a front occupant is reduced by 61% when using seat belts. In another study, Cummings [13] compared the seat belt effectiveness estimations based on police reported data and data obtained through trained crash investigators. Conditional Poisson regression method was used to estimate the risk ratios for front seat passengers using data from CDS database from 1988 to 2000. The results showed that the estimated seat belt effectiveness values based on police reported data were not substantially different from those values which were estimated based on data from crash investigators, since both estimated values were equal (relative risk of 0.36).

Johnson and Walker [14] used logistic regression method to estimate seat belt effectiveness. They used data from Crash Outcome Data Evaluation System (CODES) for seven states that participate in the CODES program. The seat belt effectiveness was controlled for factors such as occupant characteristics (age, gender), type of occupant (driver, passenger), location of crash (rural, urban), crash type, speed limit, etc. The injury severity was considered in 4 different levels: died (level 1), died or inpatient (level 2), died, inpatient, or transported (level 3), and any injury (level 4). The study found that seat belts are 89% effective in preventing fatalities (level 1) and 52% effective for any injury (level 4).
Rivara et al. [15] also used multiple logistic regression method to estimate effectiveness of automatic shoulder belt system. The seat belt effectiveness was controlled for effects of factors such as occupant age and gender, principle direction of force, automobile model year, change in the speed during the crash, and air bag deployment. The estimations were based on data from Crashworthiness Data System (CDS) for the period of 1993 to 1996. They found that effectiveness of automatic shoulder belts alone (without lap belt) reduces the fatality risk by 29% in frontal crashes and 34% reduction in all types of crashes.

Based on review of the literature, it can be seen that many previous studies have used double pair comparison method and other matched-pair analysis techniques to estimate seat belt effectiveness. However, these methods, especially the double pair comparison method have been criticized due to the difficulty in considering the effects of other variables on seat belt effectiveness. Evans and Frick [16] examined the effect of accident, vehicular and environmental factors on seat belt effectiveness against fatalities using the double pair method. They found that most of the considered factors did not have any effect on the estimated seat belt effectiveness values. Thus, this study used double-pair comparison method to estimate the seat belt effectiveness in preventing fatal and nonfatal injuries to motor vehicle occupants.

3. Methods and data

3.1 Data

The data used in this study to estimate seat belt effectiveness were obtained from Kansas Accident Reporting System (KARS) database for the period from 1993 to 2002. KARS database includes all data related to police-reported crashes occurred on public roads in Kansas. From the original KARS database, data related to vehicles that were occupied by at least two front seat occupants (i.e. driver and front right passenger) at the time of the crash were extracted. Only three vehicle types were considered: cars, vans, and pickup. Due to the limited data availability for occupants in vans, especially for fatal crashes, pickup trucks and vans were combined together and treated as a single vehicle group. Therefore, the seat belt effectiveness estimations were based on occupants in two vehicle groups: passenger cars and other passenger vehicles (pickup trucks and vans). Data related to crashes involving pedestrians, bicyclists, motorcycles and trains were discarded from the total dataset. In Kansas, the seat belt law for occupants younger than 15 years of age is a primary law compared to a secondary seat belt law for occupants 14 years of age or older, which could make a difference in seat belt usage behavior. Thus, occupants younger than 15 years of age were not considered in the estimation process. The final dataset contained crash data related to single-vehicle and multi-vehicle crashes.

In the KARS database, information related to seat belt usage was reported in 3 categories: shoulder and lap belts, shoulder belt only, and lapbelt only. In addition, unknown and none-used categories were also included where records with unknown restraint uses were discarded from the dataset used in the analysis. The effect of air bags on seat belt effectiveness was not considered in this study due to the unavailability of data in the KARS database related to air bag deployment. Based on the final dataset, the average seat belt usage rates among front seat occupants in the selected vehicle groups are shown in Table 1. It can be seen that seat belt usage among occupants involved in fatal crashes is significantly lower compared to seat belt usage among occupants in nonfatal crashes. The occupant injuries were reported using KABCO injury scale in the KARS database. The overall crash severity was defined based on the highest reported injury severity received by an involved occupant.
Tab. 1 - Seat Belt Usage by Front Seat Passengers based on KARS data from 1993 to 2002

<table>
<thead>
<tr>
<th>Crash Type (Severity)</th>
<th>Occupant</th>
<th>Type of Seat Belt Used</th>
<th>Total Involved</th>
<th>Seat Belt Usage Rate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lap belt Only</td>
<td>Shoulder belt only</td>
<td>Lap &amp; Shoulder belts</td>
</tr>
<tr>
<td>Fatal</td>
<td>Driver</td>
<td>30</td>
<td>15</td>
<td>1,637</td>
</tr>
<tr>
<td></td>
<td>Front Right Passenger</td>
<td>12</td>
<td>4</td>
<td>574</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>42</td>
<td>19</td>
<td>2,211</td>
</tr>
<tr>
<td>Incapacitating</td>
<td>Driver</td>
<td>309</td>
<td>46</td>
<td>13,440</td>
</tr>
<tr>
<td></td>
<td>Front Right Passenger</td>
<td>82</td>
<td>15</td>
<td>3,826</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>391</td>
<td>61</td>
<td>17,266</td>
</tr>
<tr>
<td>Non-incapacitating</td>
<td>Driver</td>
<td>2,279</td>
<td>171</td>
<td>84,232</td>
</tr>
<tr>
<td></td>
<td>Front Right Passenger</td>
<td>538</td>
<td>57</td>
<td>21,698</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>2,817</td>
<td>228</td>
<td>105,930</td>
</tr>
<tr>
<td>Possible</td>
<td>Driver</td>
<td>2,646</td>
<td>191</td>
<td>122,802</td>
</tr>
<tr>
<td></td>
<td>Front Right Passenger</td>
<td>558</td>
<td>34</td>
<td>29,912</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>3,204</td>
<td>225</td>
<td>152,714</td>
</tr>
<tr>
<td>No Injury (PDO)</td>
<td>Driver</td>
<td>11,200</td>
<td>537</td>
<td>554,561</td>
</tr>
<tr>
<td></td>
<td>Front Right Passenger</td>
<td>2,720</td>
<td>115</td>
<td>139,596</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>13,920</td>
<td>652</td>
<td>694,157</td>
</tr>
</tbody>
</table>

For example, if at least one of the occupants was fatally injured due to the crash then that crash was defined as a fatal crash. Based on this criterion, the total dataset was split into 5 different datasets. These categories included: fatal, incapacitating, non-incapacitating, possible, and property-damage-only (PDO). The idea was to capture any possible effects due to different level of risks involved in each crash category. For example, two occupants, who were recorded to have the same level of personal injury severity, but involved in two different crash categories, may not experience the same level of risk. By considering these two occupants in two different crash categories would minimize any bias in the estimated seat belt effectiveness.

Since there were no injuries involved in PDO crashes, the dataset related to PDO crashes was discarded. In the selected 4 datasets, the fatal crash category included occupants with all 5 types of injury severities, non-incapacitating crashes contained 4 injury severities except fatalities, incapacitating category had 3 injury types, and possible injury crashes only contained occupants with minor injuries and no injuries.
### Tab. 2 - Double Pair Estimation of Seat Belt effectiveness

Hypothetical Data Used to Illustrate the Double Pair Method

<table>
<thead>
<tr>
<th>Category</th>
<th>No. of Driver Fatalities</th>
<th>No. of Front Right Passenger Fatalities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driver Restrained, Front Right Passenger Unrestrained</td>
<td>d</td>
<td>e</td>
</tr>
<tr>
<td>Both Unrestrained</td>
<td>m</td>
<td>n</td>
</tr>
</tbody>
</table>

### Estimation of Seat Belt Effectiveness

<table>
<thead>
<tr>
<th>Subject Occupant</th>
<th>Fraction of Actual Fatalities</th>
<th>Estimated Effectiveness (%)</th>
<th>% of Fatalities Prevented</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driver</td>
<td>a</td>
<td>$E_D$</td>
<td>C = a * $E_D$</td>
</tr>
<tr>
<td>Front Right Passenger</td>
<td>b = (1-a)</td>
<td>$E_P$</td>
<td>D = b * $E_P$</td>
</tr>
<tr>
<td>Total</td>
<td>1</td>
<td>$E = C + D$</td>
<td></td>
</tr>
</tbody>
</table>

3.2 Method

In double pair comparison method, the seat belt effectiveness is estimated by comparing injury risk to a subject occupant with injury risk to another occupant in the same vehicle. Thus this method is applicable to cases where there were at least two occupants in the vehicle at the time of the crash and, at least one occupant was injured due to the crash. Since the method compares two passengers in the same vehicle, it reduces the effects of other variables which are common to both occupants. Such variables include speed of the vehicle, vehicle type and make, vehicle age, and crash type. This section consists of a brief description about the rationale behind the double pair comparison method. More detailed description can be found in publications by Evans [7,8].

To illustrate the method, the hypothetical dataset related to driver and front right passenger fatalities in Table 2 is used. In this illustration, the passengers are disaggregated only by seat belt usage. For other injury severities the procedure is similar.

The procedure starts with the estimation of fatality risk ratios between restrained and unrestrained drivers to unrestrained passengers. The ratio between restrained drivers and unrestrained passengers ($r_1$) is estimated as,

$$r_1 = \frac{d}{e} \tag{1}$$

Similarly, the ratio of unrestrained drivers to unrestrained passengers, $r_2$ is given by,

$$r_2 = \frac{m}{n} \tag{2}$$

where, variables $d$, $e$, $m$, and $n$ are defined as given in Table 2.

By using $r_1$ and $r_2$, the restrained driver to unrestrained driver fatality ratio, $R_1$ is estimated as,

$$R_1 = r_1 / r_2 \tag{3}$$

The standard error in the estimate of $R_1$, denoted by $\Delta R_1$ is given by,

$$\Delta R_1 = R_1 \sqrt{\sigma^2 + 1/n + 1/d + 1/m + 1/e} \tag{4}$$

where $\sigma^2$ is an estimate of the intrinsic uncertainty and assumed to be equal to 0.1 [8].
Similarly, by comparing restrained and unrestrained drivers with restrained passengers, the fatality ratio between restrained and unrestrained drivers, $R_2$ can be estimated. The weighted average of the ratio between restrained and unrestrained drivers denoted by, $\bar{R}$ can be estimated using the following equation.

$$\bar{R} = \exp \left[ \sum_{i=1}^{2} \frac{(R_i / \Delta R_i)^2 x \log(R_i)}{\sum_{i=1}^{2} (R_i / \Delta R_i)^2} \right]$$  \[5\]

The standard error of the estimate, $\Delta \bar{R}$ is given by,

$$\Delta \bar{R} = \frac{R}{\sum_{i=1}^{2} (R_i / \Delta R_i)^2}$$  \[6\]

Finally, the seat belt effectiveness for drivers, $E_D$ can be estimated by,

$$E_D (\%) = 100(1 - \bar{R})$$  \[7\]

Similarly, the seat belt effectiveness for front right passengers, $E_p$ can be estimated. Using these two values and proportion of actual fatalities for each occupant group, the overall seat belt effectiveness, $E$ can be obtained as shown in Table 2. The standard error of the overall estimate was taken as the standard error of the effectiveness estimation for drivers [8].

### 4. Results and discussion

The double pair estimation procedure of seat belt effectiveness for fatal injuries for passenger car occupants is shown in Table 3 and 4. It can be seen from Table 3 that the seat belts are almost equally effective in reducing fatal injuries to drivers and front passengers in passenger cars as the estimated effectiveness values for drivers and front seat passengers are 53% and 54% respectively.

The summary of estimated seat belt effectiveness values with the corresponding errors of estimates are shown in Table 5. According to the results, the effectiveness of seat belts in preventing fatal injuries to front seat occupants in passenger cars is 53% with an estimated error of 10%. In other words, 53% of fatally injured front seat occupants, who were unrestrained at the time of the crash, could have survived if all of them were restrained. Seat belts reduce incapacitating injury risk by 52% to front seat occupants in passenger cars. It can be seen that, seat belts are almost equally effective in preventing fatal and incapacitating injuries for passenger car occupants. Seat belt effectiveness for non-incapacitating injuries is 42% in passenger cars while seat belts are 34% effective in preventing possible injuries to passenger car front seat occupants.

According to estimations, seat belts reduce fatal injury risk by 57% to front seat occupants in other passenger vehicles, which is less than the NHTSA’s value of 60% for light trucks and vans. In addition, seat belts are 52% effective in reducing incapacitating injuries and 51% effective in reducing non-incapacitating injuries in other vehicle group. The seat belt effectiveness for possible injuries in this vehicle group is 28%.
Tab. 3 - Seat Belt Effectiveness against Fatalities for Drivers and Front Passengers in Passenger Cars

<table>
<thead>
<tr>
<th>Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driver Res.¹</td>
</tr>
<tr>
<td>FRP Unres.²</td>
</tr>
<tr>
<td>FRP Res.</td>
</tr>
<tr>
<td>FRP Unres., Driver Res.</td>
</tr>
<tr>
<td>FRP Res., Driver Unres.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Subject Occupant</th>
<th>Actual Fatalities</th>
<th>Fraction of Actual Fatalities</th>
<th>Estimated Effectiveness (%)</th>
<th>% of Fatalities Prevented</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driver</td>
<td>3,003</td>
<td>0.80</td>
<td>53</td>
<td>C = 42 (0.8*53%)</td>
</tr>
<tr>
<td>Front Right Passenger</td>
<td>770</td>
<td>0.20</td>
<td>54</td>
<td>D = 11 (0.2*54%)</td>
</tr>
<tr>
<td>Total</td>
<td>3,773</td>
<td>1</td>
<td>53</td>
<td>E = 53</td>
</tr>
</tbody>
</table>

Tab. 4 - Overall Seat Belt Effectiveness against Fatalities in Passenger Cars

<table>
<thead>
<tr>
<th>Subject Occupant</th>
<th>Actual Fatalities</th>
<th>Fraction of Actual Fatalities</th>
<th>Estimated Effectiveness (%)</th>
<th>% of Fatalities Prevented</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driver</td>
<td>3,003</td>
<td>0.80</td>
<td>53</td>
<td>C = 42 (0.8*53%)</td>
</tr>
<tr>
<td>Front Right Passenger</td>
<td>770</td>
<td>0.20</td>
<td>54</td>
<td>D = 11 (0.2*54%)</td>
</tr>
<tr>
<td>Total</td>
<td>3,773</td>
<td>1</td>
<td>53</td>
<td>E = 53</td>
</tr>
</tbody>
</table>

Tab. 5 - Estimated Seat Belt Effectiveness Values

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Effectiveness (%)</th>
<th>(Error) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Cars</td>
<td>53 (14)</td>
<td>52 (11)</td>
</tr>
<tr>
<td>Other Passenger Vehicles</td>
<td>57 (18)</td>
<td>47 (14)</td>
</tr>
</tbody>
</table>

* Overall effectiveness without considering the effect of air bags
The errors of estimated seat belt effectiveness values for passenger cars are lower compared to those errors for other passenger vehicles. This may be due to the smaller sample size of other passenger vehicles as compared to passenger cars.

Although state crash data was used to estimate the seat belt effectiveness in this study, there may be some concerns regarding the accuracy of police reported crash data related to variables such as injury severity and seat belt usage. This may affect the accuracy of the estimated seat belt effectiveness. According to Table 1, the reported seat belt usage in incapacitating crashes is about 75% and in possible injury crashes it is 86%, which are higher than the Kansas observed seat belt usage rates during that period of time. Reason for this over-reporting of seat belt usage may be the occupants’ unwillingness to disclose the truth to prevent any adverse consequences such as increased insurance premiums, fines, etc. The over-reported seat belt usage may result in higher estimated seat belt effectiveness than the actual effectiveness. For example, an unharmed occupant, who is incorrectly reported as restrained but was unrestrained at the time of the crash, tend to falsely over-estimate the seat belt effectiveness. Therefore, the over-reported seat belt usage in low severity crashes may result in biased estimations of seat belt effectiveness. However, data related to seat belt use in fatal crashes, in which at least one dead occupant is involved, could be expected to be more accurate [12].

The accuracy of the police reported KABCO injury severities are sometimes criticized for accuracy over AIS injury severities, which are reported by experienced medical officials at a hospital. Especially, in cases of nonfatal injuries the police officer at the scene has to decide and report the level of injury severity, which may be different from hospital reported injury severity based on thorough medical examinations by experienced medical officers. In addition, those reported severities may be subjective due to the differences in individuals’ personnel judgments. For example, Shinar et al. [17] found that injury severity is one of the least reliable variables in police reported data and sometimes could mislead the users.

Therefore, further research work would be necessary to improve the seat belt effectiveness values estimated in this study. Those improvements could be made by adjusting the estimated seat belt effectiveness values for over-reported seat belt usage. For this purpose, a survey could be conducted to collect information from the involved occupants about their seat belt usage at the time they were involved in the crash. This information could be expected to be more accurate than police reported data. Based on this information, necessary adjustment factors could be estimated. In addition, the use of more accurate injury severities could also improve the accuracy of seat belt effectiveness values. One possibility of evaluating police reported injury severities would be to compare those injury severities with hospital records.

6. Conclusions

The double pair comparison method was used in this study to estimate the seat belt effectiveness in preventing fatal and nonfatal injuries to motor vehicle occupants involved in crashes. Two vehicle groups were considered: passenger cars and other passenger vehicles (vans and pickup trucks). Based on the estimations, seat belts are 53% effective in preventing fatal injuries to front seat occupants of passenger cars. In other passenger vehicles, seat belt effectiveness in reducing fatal injuries was 57%. Both of these values are different from NHTSA’s estimated seat belt effectiveness values. Seat belts are 52% and 42% effective in reducing incapacitating and non-incapacitating injuries in passenger cars, while they are 34% effective for possible injuries. Seat belts are 47% effective in reducing incapacitating injuries in other passenger vehicles. Seat belts reduce non-incapacitating injury risk by 42% in the other
passenger vehicles while their effectiveness in preventing possible injuries is 28% in that vehicle group. There are some concerns regarding the accuracy of estimated seat belt effectiveness values due to the accuracy of the data used. However, seat belt effectiveness values based on state police reported crash data would be very useful for local highway safety agencies in evaluating highway safety programs and policies.

References
Technical note

Mobile phone use while driving and risk of road traffic injury: applying the Lorenz Curve and associated Gini Index

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Abstract

The objective of this study is to analyze the road traffic injuries according to selected risk factors based upon frequency of mobile phone use by applying the Lorenz curve and the Gini index. This is a prospective case-control study which included 217 cases and 217 control drivers. A structured questionnaire was designed to collect demographic information like age, gender, marital status, occupation, residential area, housing conditions, driving experience, type of car, frequency of seatbelt usage, excessive speed and other violations and frequency of mobile phone use. The Lorenz curve and the associated Gini index are applied for characterizing and testing road traffic injuries stratified by mobile phone use. We also showed that the Gini index can itself be interpreted as a probability related to temporal clustering. In conclusion, with the increasing demand for computer technology the proposed method is well-suited for analyzing RTI data.

Keywords – Road Traffic Injuries, Mobile Phone use, Driver Behaviour, Lorenz Curve, Gini Index, Qatar

1. Introduction

It was estimated that in 2002, road crashes killed 1.18 million people and injured about 20 to 50 million more worldwide [1]. More than 3,000 die daily as a result of road crash while 140,000 people are injured and about 15,000 are disabled for life. It is predicted that by the year 2020, the annual number of deaths and disabilities from road traffic injuries will increase by 60% or more, and has taken third place in the rank order of burden diseases by the year 2000 as compared with the ninth in 1990 [1]. Road traffic safety has become a neglected issue in Qatar, where majority of the victims were young men and it has already taken third rank in leading causes of death since 2002 [2].

More recently it has been reported that Lorenz curve and the Gini index are excellent alternative to relative risk and attributable risk in evaluating the impact of a public health measure or an intervention to a different population with different distributions of risk.

The Lorenz curve is widely used by economists and demographers to assess the distributional properties of wealth and to quantify the degree of population concentration however, its applications in the field of epidemiology are continuously noticed in recent literature by statisticians and clinicians [3,4].
Observational surveys found that around 5% of drivers at any given time were operating handheld mobile phones [5] which is a major cause of distraction [6] and increases the possibility of road crashes [7,8]. The objective of this study is to analyze the road traffic injuries according to selected risk factors based upon frequency of mobile phone use by applying the Lorenz curve and the Gini index.

2. Subjects and statistical methods

A prospective case-control study was conducted between October 2004 and March 2005 to examine the differences between road traffic crash victims and those who had no previous history of road traffic crashes.

A total of 600 Qatari drivers were approached in two different locations. Of these, 434 drivers who expressed their consent were recruited to participate in this study with a response rate of 72.3%. Subjects were randomly selected and assessed for their eligibility before contacting to invite them to participate. Study was based on Qatari men and women who were above 18 years of age who were either defined as case or control. Among the recruited drivers (434), 217 drivers were selected as cases from the Accident and Emergency department after checking the medical records.

Apart from also being a university hospital, Accident and Emergency department is a level 1 trauma center or tertiary referral center. A case was defined as any subject who visited the Accident and Emergency Department for treatment following a road traffic crash in which he/she was involved as a driver. For control subjects, eleven Primary Health Centers (PHC) were selected randomly, eight urban and three semi-urban area from different geographical areas in the State of Qatar. The control subjects (217) were randomly selected from these eight PHCs who did not have any road traffic crash history. A control was defined as subjects who were drivers and who did not have any road traffic crash history.

A structured questionnaire was designed to collect the demographic information like age, gender, marital status, occupation, residential area, housing conditions, driving experience, type of car, frequency of seatbelt usage, and violations like speeding, crossing the red signal along with frequency of mobile phone use. Both the cases and controls were interviewed face to face by well trained nurses.

3. Statistical method and analysis

Lorenz Curve and the Gini Index:

If we assume the risk factors for road traffic crashes for example with k levels which is being arranged from low to the highest and are indexed by i, (i=1,….k), the number of subjects in each exposure level is represented by ni and the number of cases by di. The Lorenz curve is constructed by plotting the empirical cumulative distribution of road traffic injuries against the cumulative distribution of controls.

If the risk of road traffic injuries at different exposure levels is equal to each other, then the Lorenz curve will be a straight diagonal line for that exposure. The greater the inequality of road traffic crashes over exposure levels, the greater the discrepancy between the Lorenz curve and the line of perfect equality. The variation depicted using a Lorenz curve can be summarized using the Gini index [9-11]. The value of the index is between 0 and 1, with larger values reflecting greater variability in road traffic crashes over exposure levels and smaller values reflect greater uniformity. One realization of the index is expressed mathematically as:
\[ Gini = \sum_{i=1}^{k} \frac{x_{i} - y_{i}}{x_{i} + y_{i}} = \sum_{i=1}^{k} (x_{i} - y_{i}) \]

where \((x_i, y_i)\) represents the coordinates of the points in the Lorenz curve,
\[ x_i = \sum_{j \in S} n_i / N \]
and \[ y_i = \sum_{j \in S} a_i / A \cdot n_i \]
denotes the incidence in the \(i^{th}\) exposure level \((i=1, \ldots, k)\) and \(N\) denotes the total incidence \((N = \sum n_i)\). \(A\) and \(a_i\) denote, respectively, the total number of road traffic injuries in the \(i^{th}\) exposure level and \(A = \sum a_i\).

4. Results

We identified 217 cases of road traffic injury who visited the Accident and Emergency department during the study period and were compared with 217 controls. The mean age of the cases was 33.4±9.7 years and that of the controls was 34.0±11.0 years with no significant difference. Among the cases were 162 males and 55 females, and among controls were 155 males and 62 females. Use of mobile phone while driving is a well known confounding variable for road traffic injuries and therefore all the exposures are stratified by this variable in Table 1. The road traffic injury was more common among young drivers who used mobile phone while driving and a similar trend was noticed among the non mobile phone users. The use of mobile phone while driving was highly significant among least educated drivers. The drivers who used mobile phone while driving were less likely to wear a seat belt than drivers who did not use mobile phone 6.66 (95% CI= 3.30-13.58). Even though the incidence of road traffic injuries was higher among mobile phone users, all exposures cause high hazard in subjects who did not use mobile phone while driving except in the case of educational level and use of seat belt.

**Lorenz curves and the Gini indices**

Using data from Table 1, Lorenz curves were constructed on selected exposures. In figures 1, 2, 3 and 4, we see Lorenz curve of non mobile phone users is always below the mobile phone user curve and covering more area showing that subjects who did not use mobile phone while driving have a greater inequality to the risk factor: speeding on road. The lesser the area between the diagonal line and the Lorenz curve, the higher the risk of road traffic injuries in that population.

![Fig. 1 - Lorenz curve of RTI and Age group by mobile phone use while driving](image-url)
Tab. 1 - Socio-demographic information of study subjects

<table>
<thead>
<tr>
<th>Variables</th>
<th>Yes</th>
<th></th>
<th></th>
<th>No</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case</td>
<td>Control</td>
<td>Odds ratio</td>
<td>95% CI</td>
<td>Case</td>
<td>Control</td>
</tr>
<tr>
<td>Gender</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Male</td>
<td>120(76.4)</td>
<td>119(77.3)</td>
<td>1.00</td>
<td>42(70.0)</td>
<td>36(57.1)</td>
<td>1.00</td>
</tr>
<tr>
<td>Female</td>
<td>37(23.6)</td>
<td>35(22.7)</td>
<td>1.05(0.60-1.84)</td>
<td>18(30.0)</td>
<td>27(42.9)</td>
<td>0.57(0.25-1.28)</td>
</tr>
<tr>
<td>Age group</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥45</td>
<td>26(16.6)</td>
<td>27(17.5)</td>
<td>1.00</td>
<td>7(11.7)</td>
<td>14(22.2)</td>
<td>1.00</td>
</tr>
<tr>
<td>35-44</td>
<td>47(29.9)</td>
<td>27(17.5)</td>
<td>1.81(0.83-3.95)</td>
<td>17(28.3)</td>
<td>17(27.0)</td>
<td>2.00(0.56-7.24)</td>
</tr>
<tr>
<td>25-34</td>
<td>58(36.9)</td>
<td>64(41.6)</td>
<td>0.94(0.47-1.89)</td>
<td>18(30.0)</td>
<td>21(33.3)</td>
<td>1.71(0.50-6.00)</td>
</tr>
<tr>
<td>&lt;25</td>
<td>26(16.6)</td>
<td>36(23.4)</td>
<td>0.75(0.34-1.68)</td>
<td>18(30.0)</td>
<td>11(17.5)</td>
<td>3.27(0.87-12.71)</td>
</tr>
<tr>
<td>Education level</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>University</td>
<td>20(12.7)</td>
<td>43(27.9)</td>
<td>1.00</td>
<td>13(21.7)</td>
<td>15(23.8)</td>
<td>1.00</td>
</tr>
<tr>
<td>Secondary</td>
<td>58(36.9)</td>
<td>71(46.1)</td>
<td>1.76(0.89-3.48)</td>
<td>18(30.0)</td>
<td>26(41.3)</td>
<td>0.80(0.28-2.31)</td>
</tr>
<tr>
<td>Intermediate</td>
<td>28(17.8)</td>
<td>21(13.6)</td>
<td>2.87(1.23-6.73)*</td>
<td>5(8.3)</td>
<td>11(17.5)</td>
<td>0.52(0.12-2.26)</td>
</tr>
<tr>
<td>Primary</td>
<td>32(20.4)</td>
<td>11(7.1)</td>
<td>6.25(2.43-16.44)†</td>
<td>16(26.7)</td>
<td>3(4.8)</td>
<td>6.15(1.25-34.18)*</td>
</tr>
<tr>
<td>Illiterate</td>
<td>19(12.1)</td>
<td>8(5.2)</td>
<td>5.11(1.74-15.43)†</td>
<td>8(13.3)</td>
<td>8(12.7)</td>
<td>1.15(0.28-4.71)</td>
</tr>
<tr>
<td>Use of seatbelt</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Always</td>
<td>19(12.1)</td>
<td>82(53.2)</td>
<td>1.00</td>
<td>9(15.0)</td>
<td>37(58.7)</td>
<td>1.00</td>
</tr>
<tr>
<td>&gt; than half of the trips</td>
<td>58(36.9)</td>
<td>13(8.4)</td>
<td>19.26(8.27-45.78)†</td>
<td>28(46.7)</td>
<td>12(19.0)</td>
<td>9.59(3.21-29.71)†</td>
</tr>
<tr>
<td>&lt; than half of the trips</td>
<td>26(16.6)</td>
<td>24(15.6)</td>
<td>4.68(2.08-10.59)†</td>
<td>2(3.3)</td>
<td>8(12.7)</td>
<td>1.03(0.13-6.85)</td>
</tr>
<tr>
<td>Never</td>
<td>54(34.4)</td>
<td>35(22.7)</td>
<td>6.66(3.30-13.58)†</td>
<td>21(35.0)</td>
<td>69(95)</td>
<td>1.25(0.48-3.30)</td>
</tr>
<tr>
<td>Speed on 100 Km/hr road</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt;100</td>
<td>35(22.3)</td>
<td>57(37.0)</td>
<td>1.00</td>
<td>9(15.0)</td>
<td>34(54.0)</td>
<td>1.00</td>
</tr>
<tr>
<td>100-120</td>
<td>71(45.2)</td>
<td>62(40.3)</td>
<td>1.86(1.05-3.33)*</td>
<td>34(56.7)</td>
<td>22(34.9)</td>
<td>5.84(2.16-16.16)†</td>
</tr>
<tr>
<td>&gt;120</td>
<td>51(32.5)</td>
<td>35(22.7)</td>
<td>2.37(1.24-4.54)†</td>
<td>17(28.3)</td>
<td>7(11.1)</td>
<td>9.17(2.56-34.64)†</td>
</tr>
<tr>
<td>Speed on 120 Km/hr road</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt;100 km/hr</td>
<td>38(24.2)</td>
<td>59(38.3)</td>
<td>1.00</td>
<td>20(33.3)</td>
<td>44(69.8)</td>
<td>1.00</td>
</tr>
<tr>
<td>120-140 km/hr</td>
<td>62(39.5)</td>
<td>52(33.8)</td>
<td>1.85(1.03-3.33)*</td>
<td>22(36.7)</td>
<td>16(25.4)</td>
<td>3.03(1.22-7.60)*</td>
</tr>
<tr>
<td>&gt;140 km/hr</td>
<td>57(36.3)</td>
<td>43(27.9)</td>
<td>2.06(1.12-3.79)*</td>
<td>18(30.0)</td>
<td>3(4.8)</td>
<td>13.20(3.13-64.16)†</td>
</tr>
</tbody>
</table>

p value< 0.05; † p value < 0.001

Fig. 2 - Lorenz curve of education level and Age group by mobile phone use while driving
5. Discussion

The present study used the data to construct the Lorenz curve and the calculation of the Gini indices. We have used the Lorenz curve and the associated Gini index to test the road traffic injuries stratified by mobile phone use for a number of reasons. Firstly, when we deal with multiple levels of the same exposure, the index of relative risk is not a common unit of comparison across different exposures. Secondly, the index of attributable risk sometimes does not adequately describe the variation of disease risk in populations. Thirdly, the advantage of the Gini index over other measures is that it is comparable across different exposures [3]. In the current study, using this interpretation of the Lorenz curve, we can state that road traffic injuries among drivers are highly concentrated among mobile phone users who have low education level and who never used seatbelt. Those who did not use mobile phone while driving show a higher inequality for exceed of set speed limit on road. There are, however, several issues concerning this method that needs to be considered. These include examining of its sampling variability, testing for hypothesis, constructing confidence interval for measures and the possibility of applying a multivariate setting to the Gini index [4]. Speeding greatly increases risk of injury and death [12]. Research on travelling speeds and crash involvement has found high risks of exceeding speed limits on injuries [13]. In our study, it is worth noting that the road traffic injury was always high when a driver exceeded the speed limit imposed on the road. Despite the clear effectiveness of seatbelts [14] and knowledge that motor vehicle crashes are a major cause of injury, non-compliance with seatbelt use in reported cases is very high. Necessary injury prevention programs are required which include seatbelt education and strict penalties for consequent offenders. Driving while operating a mobile phone can be distracting and potentially dangerous and driving performance is hindered significantly and more likely to lead to road traffic
In fact, driver distraction is an important cause of crash involvement [5], with mobile phones being one device known to distract drivers. Most recently study in the Spanish university workers [8] reported that the frequency of using mobile phone to talk on urban roads was significantly correlated with crash involvement. Furthermore, several studies reported when drivers use a mobile phone there is an increased likelihood of a crash resulting in injury, using a hands-free phone is not any safer [6-8,15-16], this is consistent and confirmed with the current study performed in the State of Qatar.

6. Conclusion

This study has helped us to analyze the road traffic injuries in drivers who used mobile phone and who did not use a phone by applying the Lorenz curve and the Gini index. The results showed that the Lorenz curve and the associated Gini index can be applied for characterizing and testing the road traffic injuries stratified by mobile phone use. There is a need for better road designs, education interventions, strict enforcement of laws and excellent emergency facilities to increase road safety and reduce severe injuries related to road traffic crashes.

References

IN OCCASION OF THE
International Conference RSS 2007              7th – 9th November 2007 Rome, Italy

Mr. Abdullah A.M. Al-Mogbel
Ministry of Transport – Kingdom of Saudi Arabia

He is Deputy Minister of Transport and Roads in the Kingdom of Saudi Arabia from 1997, he has been for about ten years supervisor engineer and general director of design and studies. This year he has been awarded as man of the year for his efforts and contributions in the sector of road safety from the International Road Federation. Mr. Al-Mogbel is affiliated to the most important international and Arabic institutions involved in transportation research. He published many scientific papers in conferences and forums all around the world and received more than forty prestigious awards and citations for his activities in the sectors of transportation.

Considering the number of death and injuries that happen on the roads, European Commission has decided to take on the ambitious target of reducing the death rate on the roads by half by 2010. How are European Member States progressing towards this objective?

The issue of road safety is definitely of great importance to the Ministry of Transport (MOT) in the Kingdom of Saudi Arabia (KSA). Some statistics report that the fatality risk associated with roadway accidents in KSA reaches 21 fatalities per 100,000 population. Further, it is estimated that, depending on the study and source, the annual cost of roadway accidents in KSA is between 2.2 to 5.5 billions dollars. Accordingly, the roadway safety is considered a priority for policy makers at all of the relative public service establishments. The reduction of the number of roadway accidents and the injuries associated with these accidents in KSA has been one of the priorities for several public and private establishments including the Ministries of Transport, Interior, Municipalities and Rural Affairs, Education, and Culture and Information, as well as King Abdul-Aziz City for Science and Technology, Traffic Management Police, Arriyadh Development Authority, and the Saudi Arabian Red Crescent. In the years 1998 and 2004, these agencies with many others held two major National Conferences on Traffic Safety. The conferences resulted in several short and long-term recommendations and action oriented plans to be undertaken by each of these agencies to reduce the number of roadway accidents and the injuries associated with them throughout KSA. Also, a high-level national committee with officials from these agencies was formed to monitor and guide the execution of these recommendations and plans. Additionally, in 2004, Arryiadh Development Authority put a strategic plan for reducing the expected number of fatalities and injuries associated with roadway accidents in the city of Riyadh for the year 2014 by 30 percent. In the recent years, KSA has been investing heavily in the design and construction of its roadway network with high emphasis on roadway safety and transportation management including the implementation of ITS technology. These investments accompanied with large investments in public transit projects and railroads are expected to alleviate some of the major roadway safety issues in KSA.

Do you think that additional financial resources should also be spent in your country to improve road safety? What should be the priorities for the most effective action to increase the safety standards for drivers, passengers and road users?

There is always some room for improvement and additional financial resources will always be needed to accomplish such improvements. Accordingly, the answer to this question is yes.
However, we all know that there are many competing factors that affect the use and distribution of financial resources. Consequently, a suitable effort should be directed toward defining the road safety problems, determining and prioritizing the contributing factors causing these problems, allocating sufficient funding for treating these factors, investing the necessary funds to treat these factors, and using objective evaluation tools to assess the benefits relative to the costs of such investments.

The research in the area of roadway safety shows that driver behaviour is a major contributing factor to roadway accidents. Accordingly, a major part of our efforts should be directed toward studies aimed at treating this factor. While many of the benefits of safety research, standards, and technologies in the design of roadways and vehicles can be transferred from one nation to another, the benefits of standard solutions for changing problematic driver behaviours are hardly transferable. Accordingly, a diligent effort should be directed at research in driver behaviour suitable for the population of drivers for which positive changes in behaviour are needed. For example, with minor differences, many of the engineering standards in the design of roadways are common among most countries. However, for a variety of reasons, drivers’ behaviours differ from one country to another. Even within the same country and/or society, drivers’ behaviours differ for a variety of factors including geography, age, and/or gender. Delineating research methods and technological tools that capture the behavioural attributes of a population of drivers within a specific country/society and, consequently, delineating objective methods for positively changing these attributes should be a priority to increase the safety of drivers, passengers, and roadway users; especially in developing countries.

Finally, in most developing countries, the public sector is the major contributor to investments in roadway safety. Attention should be directed at encouraging the private sector to increase their investments in this area by creating suitable environment for the private sector to capitalize on such investments. In part, this can be achieved through the introduction of suitable legislation and better traffic law enforcement.

**RSS2007** overview the state-of-the-art research and technologies concerning key themes in safety and simulation, such as roadway design, emerging technologies and human factors. What the expected outcomes and impacts of this interdisciplinary conference could be?

**RSS2007** will be a great opportunity for researchers and practitioners (including policy makers) in different fields of science and technology related to roadway safety to share knowledge, experiences, and skills and to exchange ideas. The interdisciplinary aspect of the conference is extremely valuable since it mirrors the nature of the field of roadway safety. In addition to mechanical, civil, transportation, and traffic engineering, in the recent decades, we have been seeing great contributions to the field of roadway safety from researchers in the fields of industrial engineering, psychology, computer engineering, biomechanics, mathematics, physics, and statistics. Because of that, to mention a few, we have made major breakthroughs in simulating driving, modelling accidents and injury mechanisms, and understanding drivers’ behaviour, abilities, and limitations. **RSS2007** will add to our knowledge in these areas and will equip us with new tools to tackle current and future roadway safety issues.
Mr. Zhongyin Guo
University of Tonji Shangai – School of Transportation – China

Professor Zhongyin Guo, PHD Student Supervisor, works at School of Transportation Engineering, Tongji University. He engages as vice-director of Road and Traffic Laboratory of Ministry of Education, and takes many part time jobs such as personal member of PIARC, Syndic of Road Safety Academy of China, Vice-director Commissioner of Road and Traffic Engineering Specialty Committee, Associate Editor of “The International Journal of Road Materials and Pavement Design” and “Advances in Transportation Studies”, etc.

He has been doing research in the field of road safety and carrying on many projects, including Operation safety enhancement countermeasures for Hang Zhou Bay Bridge in bad weathers, Safety enhancement system for high-grade highways in Xinjiang Province, Safety of the entrance and exit zone of highway tunnels, Safety management countermeasures for the operation of Hurongxi freeway, etc.

Considering the number of death and injuries that happen on the roads, European Commission has decided to take on the ambitious target of reducing the death rate on the roads by half by 2010. How are European Member States progressing towards this objective?

Road accidents and their consequences constitute a serious problem in many countries. China exhibits substantially higher accident rates than average. In spite of such statistics, the systematic safety management technology has not yet been established and implemented, especially in developing countries. In recent years, safety attains increasing consideration among road management agencies under public pressure for a safer road environment. Given the importance of reducing the social and economic costs associated with collisions, most road authorities employ some type of road safety management program, designed to improve the road safety performance for the system users. In May 1st, 2004, the first edition of road traffic safety law was implemented in China, which contributes greatly to constrain and standardize the human behavior in road traffic system. What’s more, the proactive road safety planning has been paid much attention in china recently.

Do you think that additional financial resources should also be spent in your country to improve road safety? What should be the priorities for the most effective action to increase the safety standards for drivers, passengers and road users?

The developing countries, including china, have been suffering serious road safety problem, causing extensive damage to transportation networks, leading to significant repair costs, emergency access constraints and disruption to users and the community. It’s necessary to spend additional financial resources to improve road safety in china. The World Bank and The Asian Bank have been supporting many projects on improving road safety in china. Road accidents are caused by several different factors. In general, they can be grouped into three broad categories: human component, vehicular component and road-environmental component. Research has shown that in most accidents the human component is the primary contributory factor to the occurrence of accidents. The road-environmental and vehicular components come, respectively, second and third in the ranking of factors that contribute to accidents. Although the
human component is responsible for the largest number of accidents, actions in this area do not always present the best cost-benefit relationship. Engineering measures aimed at improving road environments is frequently more cost effective and easier to implement than training drivers to deal with complex environments. Road infrastructure is not only the important components of road environment, but easier to control, compared with other components such as climate and geography conditions. Road infrastructure with good safety performance contributes to a high level of safety.

RSS2007 overviews the state-of-the-art research and technologies concerning key themes in safety and simulation, such as roadway design, emerging technologies and human factors. What the expected outcomes and impacts of this interdisciplinary conference could be?

With active participation of many prestigious experts and researchers dealing with road safety study, this interdisciplinary conference will provide a good opportunity for them to exchange research results and creative ideas. The state of the art research and technologies will be worldwide spread and the further research will be highlighted.
IN OCCASION OF THE
International Conference RSS 2007 7th – 9th November 2007 Rome, Italy

Mr. Alfredo Garcia Garcia
Polytechnic University of Valencia – Spain

Director of the Department of Transportation at the Polytechnic University of Valencia (Spain)
Member of the following committees
- AHB65 TRB Committee Operational Effects of Geometrics
- PIARC-Spain Comm. C3.1 Road Safety
- PIARC-Spain Comm. C3.2 Risk Management for Roads

Member of the Editorial Board for the international journal “Advances in Transportation Studies”
Member of the Editorial Board for the Spanish journal “Rutas” of the PIARC-Spain.

Considering the number of death and injuries that happen on the roads, European Commission has decided to take on the ambitious target of reducing the death rate on the roads by half by 2010. How are European Member States progressing towards this objective?

The European goal is very ambitious but necessary to reduce the high number of deaths every year and therefore the great number of crashes and injuries. As infrastructure deficiencies being a major killer on European roads, the Commission’s proposal for a Directive on road infrastructure safety management is the only right way. It offers member states a chance to take action to significantly reduce the number of victims on their roads. All road safety experts and professionals across Europe unequivocally welcome this proposal, which require member states to develop guidelines for the implementation of four policy instruments that are known to bring effective and cost-efficient safety benefits: road safety impact assessment; road safety audit; network Safety Management; and road safety inspection. The proposal does not impose any technical standards or procedures but invites member states to make use of these instruments leaving the details of their implementation to the member states. Despite, it might represent a major bureaucratic burden, the objective is worthwhile.

In Spain the safety problems is very important, since the accidents provoke about ten percent of all fatalities around Europe and the accident rates are below European average ones. The EU actions program target to halve the number of fatalities in road accidents by 2010, from data in 2001, has reached an insufficient result until 2006. Spain has improved in the same period from 5,517 fatalities to 4,104 fatalities, that means a 25.6% decrease; a little less than the target. These data bring Spain closer to the average in Europe demonstrating with time that traffic accidents are avoidable and encourage us to go on working to consolidate on this line of improvement. After the first year of the new Penalty Point Driving Permit the favourable tendency goes on.

The new driving licence may have contributed to higher driver awareness on road risks and to subsequently change their behaviour. Moreover, persistent special campaigns, stressing on road safety key points, like the use of seat belts, crash helmets, drinking and alcohol and over speeding may as well lay behind the improvement in the figures. Despite the existing freeways and double carriageways which are in total a little less than 15,000 km, about 77% of fatalities that took place on roads, are located along two lane rural roads.
The main road network has significantly been improved and now the safety objective must be concentrated in single carriageway roads.

Do you think that additional financial resources should also be spent in your country to improve road safety? What should be the priorities for the most effective action to increase the safety standards for drivers, passengers and road users?

Yes, the more financial resources the less accidents. But to contribute in the taking of decisions to tackle traffic accidents in our roads a strong-willed safety strategy that it is paramount. When accidents happen, drivers are blamed for the mishap. When drivers consistently fail at certain locations, it then becomes obvious that the problem lies not only with them, but with the geometric design and facilities of the road itself. Thus, actions on the human factor should be continued and enhanced, with an adequate enforcement and constant particular campaigns, but improvements in the design, construction and operation of the roads must be achieved. All road administrations should not hide their safety problems, but communicate and recognise the ways to correct them in a short time if possible. This should be the best way to express our compromise with safety.

RSS2007 overviews the state-of-the-art research and technologies concerning key themes in safety and simulation, such as roadway design, emerging technologies and human factors. What the expected outcomes and impacts of this interdisciplinary conference could be?

The driver behaviour has a main influence on road safety but its research is always complicated. New technologies, including interactive driving simulators, let us correctly research human factors or resulting roadway design. Driving simulators and traffic simulations facilitate under controlled and repeatable conditions, monitoring in real time, an enormous number of experiments or observations at relatively low costs. These objectives and approaches should be developed in an interdisciplinary way, and the conference let researchers and practitioners met their advances, the state of the research area, the problems being challenge and the objectives for future efforts.
Mr. Brendan Halleman
European Road Federation – ERF – Bruxelles

An economist by training, Brendan Halleman has been Director of Operations at the European Union Road Federation (IRF Brussels Programme Centre) supervising the Federation’s research and technical activities since 2002. His areas of expertise include road equipment standards, safe road design & management, infrastructure financing and GNSS applications in the road sector.

Considering the number of death and injuries that happen on the roads, European Commission has decided to take on the ambitious target of reducing the death rate on the roads by half by 2010. How are European Member States progressing towards this objective?

The figures for 2005 indicate that approximately 41,500 road users lost their lives throughout the EU25, a 17.5% drop over the four years since the 50% target was introduced. More spectacular, the number of accidents fell by 5% between 2003 and 2004, which corresponds to the date of entry into force of voluntary road safety plans in most of the Member States. Though encouraging, these figures suggest that the target of a maximum of 25,000 fatalities will not be reached by 2010 and conspicuous problems remain. Some user categories, such as motorcyclists and novice drivers, are at higher risk than others. Above all, the road safety performance of many new Member States following the 2004 enlargement is still not as good as the average situation in the EU-15, and in some cases is worsening.

Do you think that additional financial resources should also be spent in your country to improve road safety? What should be the priorities for the most effective action to increase the safety standards for drivers, passengers and road users?

Despite the relentless drive towards harmonisation of speed limits, blood alcohol levels and enforcement policies, the fact remains that some EU countries – and some roads – are far safer to drive in than others. In fact, even in European Member States where identical speed limits and blood alcohol levels are applied, the road death ratio stands at 1:3. Such disparities are a clear indication that Member States need to look beyond the traditional "human pillar". The simple truth known to every motorist is that safer roads save lives, every day. The physical layout of the road provides visual cues and signals for motorists to adapt their driving speeds while adequate roadside protection mitigate the consequences of run-off accidents - thought to kill 16,000 motorists every year. These and other statistics previously researched and published by ERF suggest that the infrastructure is responsible for up to a third of all casualties. Despite the obvious advantages and cost-effectiveness of a preventive approach to safe road management, European roads – and the Trans-European Network is no exception – offer a patchwork of safety standards where unprotected trees, surface defects and road signs that cannot be seen at night or under rain are all too common features. It is fair to say that motorists are the first victims of poorly planned, designed, signposted and maintained road networks. Key decisions on how to build a new road and where to affect maintenance funds are often made without a clear understanding of their safety implications. How else can one explain that, after
decades of under-investments, funds allocated to structural maintenance still only reach 50% of required expenditure and are regularly poached by other, more attractive budgets? The result is that highways engineers across Europe are fighting a losing battle to keep the roads fit for their intended purpose.

**RSS2007 overviews the state-of-the-art research and technologies concerning key themes in safety and simulation, such as roadway design, emerging technologies and human factors. What the expected outcomes and impacts of this interdisciplinary conference could be?**

RSS 2007 looks at the crucial link between roadway design and human factors, making it one of the most relevant conferences for the road research community. In many cases, responsible, law-abiding drivers die on Europe’s roads because they unexpectedly face a momentary situation with which they cannot cope. Driving errors and accidents demonstrate the limits of drivers’ adaptation to their lane-keeping task, and the factors responsible for that need to be analysed, understanding the reasons for such deviations, identifying the conditions in which they are most likely to appear and analysing the mechanisms that could explain their occurrence. We know that the individual accident is an unpredictable event, but we also know that accidents as an aggregate are systematically over-represented at certain locations and in certain circumstances. RSS 2007 will contribute to a better understanding of the human physiological and psychological abilities, limitations and needs of road drivers and the complex interaction between the key components of the road transport system. For all these reasons, RSS 2007 provides a natural framework for the special RANKERS session on Friday 9 November.
Mr. Basil Psarianos
Technical University of Athens – Greece

Considering the number of death and injuries that happen on the roads, European Commission has decided to take on the ambitious target of reducing the death rate on the roads by half by 2010. How are European Member States progressing towards this objective?

In our case more than it can be generally expected. Greece has one of the worst road accident records in the EU. Even the EU set aim of halving the road accidents proves to be a very low rate for Greece. We have achieved some improvements between 2001 and 2003 but the total number of fatalities remain almost constant in the last 3 years despite the fact that we have constructed and put into service almost 1500 km of motorways; a fact that alone reduces accidents significantly. We have to impose much higher rates of crash reduction in order to be able to reach accident records like those of the U.K., Sweden, the Netherlands or France and Germany in the not so far future. Therefore the Technical Chamber of Greece, which represents by law all Greek Engineers having understood the necessity for a combined and concerted technical action on road safety issues in my country has established a team of road engineers working constantly and on a continuous basis for improving road safety in Greece. This team of engineers is in action now for 2 years and it is hoped that the too many odds which govern the unsafe road conditions in my country will be diminished in the next years thus helping the country improve the road safety records significantly in the near future.

Do you think that additional financial resources should also be spent in your country to improve road safety? What should be the priorities for the most effective action to increase the safety standards for drivers, passengers and road users?

In order to achieve safety levels that are comparable with those of the so called "road safe countries" in the EU we have to take a considerable amount of measures with regard to all parameters of the mobility issue, i.e. drivers and passengers, infrastructure, vehicles and law enforcement. We need a better education and mobilization of drivers with regard of accepting and being more sensible to safe driving attitudes, we need to introduce a quality assurance process for designing, constructing or reconstructing and rehabilitating state-of-the-art roadways with respect to road safety, which is missing in my country. I know of cases where technical departments overseeing and responsible of 1500 km of national highways do not have in their staff not even one transportation engineer. I wonder how can such a department assess its national roadway
network in terms of safety and necessary measures to improve the existing bad road safety levels. We also have to take measures for renovating our vehicle fleet since too many old vehicles are still in traffic on Greek roadways. Additionally we must find ways of better enforcing basic safe driving attitudes and habits of our drivers. This can only be achieved by increasing both the number of police officers overviewing our road network and introducing new technological methods and equipment for a more efficient and effective law enforcement.

RSS2007 overviews the state-of-the-art research and technologies concerning key themes in safety and simulation, such as roadway design, emerging technologies and human factors. What the expected outcomes and impacts of this interdisciplinary conference could be?

I personally look forward to such an important event from the point of view of an expert. International research and engineering experience on a wide spectrum of road safety issues presented in a compact way like the RSS2007 will help every participant from all parts of the world gain the utmost informational bulk of knowledge which he or she will take back at home. Especially I find very useful the interdisciplinary approach of all road safety arising issues. Road safety can not be improved by isolated measures only. All actors and stakeholders of road safety have to combine technical skills, research findings, technological outcomes, ideas and perspectives in order to achieve the best safety outcome for all road users. We need to imply always the best available knowledge for the appropriate measure at the correct time with the higher benefit/cost ratio possible to effectively counteract unsafe road operation conditions. The information that I expect to gain from RSS2007 will be further disseminated to various administration officials in my country, will enrich my lectures at the National University of Athens, will help me define problem statements and finally guide me develop my research initiatives in the next years, which I hope to be able to present at the next RSS. Therefore I will like to thank all initiators and organizers of such an important technical and scientific event.
Mr. Michael Manore & Richard Pain  
Transportation Research Board – USA

He is currently Director of the newly formed Infrastructure Visualization Center who’s charter is to research, prototype, and advance Bentley’s offerings in the realm of visually enhanced infrastructure delivery. Michael’s past responsibilities include business development for Bentley solutions focusing on the Transportation Operations & Management industries, technology sales, and consulting engineering to government and transportation clients. He also has extensive public sector experience with the FHA, Wisconsin DOT and Minnesota DOT.

His relevant accomplishments include: Initiating and managing Minnesota DOT’s first Visualization Assessment and Development Project, Chairman of TRB Visualization Committee (1996 – present), Chairman of NCHRP Human Factors Guide Research Sub-Committee, Panel Member NCHRP Synthesis 361 – Visualization for Project Development

Considering the number of death and injuries that happen on the roads, European Commission has decided to take on the ambitious target of reducing the death rate on the roads by half by 2010. How are European Member States progressing towards this objective?

Yes, in the United States, Safety has become a leading issue for roadway transportation agencies, and in recent years, the United States has fallen behind globally with respect to the number of fatalities per total vehicle miles travelled. According to the Federal Highway Administration [http://safety.fhwa.dot.gov](http://safety.fhwa.dot.gov), 43,443 fatalities occurred in 2005 broken down into the following categories: 25,347 – Road Departures; 9,188 – Intersection Related; 4,881 – Pedestrians. And despite improvements in vehicle safety design, and stricter laws regarding impaired driving and speed enforcement, the U.S. have seen a recent increase in overall fatalities. In 2006, the Transportation Research Board published its “Critical Issues in Transportation” report which also recognized Safety as a leading concern. As all of us in the transportation community know, safety involves a number of factors, but the primary components are the Vehicle, Driver, and the Roadway. There have been a number of activities over the years to improve performance and crash-worthiness and performance of the vehicle, as well as improving the training and education of drivers, with enhancements to these components continually evolving through new technologies, training, and information dissemination. With Pedestrians accounting for 13% of the overall transportation fatalities in 2005, there are efforts to improve education and awareness for this mode as well. But we are now also digging much deeper into Roadway (infrastructure) component to perform improved consideration and analysis with respect to:


The important message here is that the United States is rethinking how we consider and improve the safety performance of the infrastructure itself, and do so more integrally with the Driver and the Vehicle in mind.
Do you think that additional financial resources should also be spent in your country to improve road safety? What should be the priorities for the most effective action to increase the safety standards for drivers, passengers and road users?

Additional financial resources are needed for transportation overall in the United States and safety is included. However, not only are additional resources needed but the rules for using existing funds need to be adjusted. For example, roughly 40-50% (varies by state) of the fatalities occur on local roads. (Those are roads owned by local government agencies such as municipalities or counties; not the state.) Yet, a major share of safety funding goes to states, not to the local agencies. There are other areas where current restrictions on use of funds limit a state or local agency from applying money to the greatest need. Hence, greater flexibility in using funds is needed. The overall answer to the second part of the question is: It Depends. This means the priorities vary from state to state and local area to local area. In very rural states run off the road is the major issue. In more urbanized states intersection issues are more dominant. From a national perspective an important priority is to continue and strengthen the requirement, instituted legislatively in the last highway reauthorization, for each state to develop a Strategic Highway Safety Plan (SHSP). The importance of this initiative is it mandates the many partners involved in highway safety develop a single plan together. In the past there have been a minimum of three separate safety plans in state and they were never integrated. The SHSP process also means the various agencies and organizations that, in the past, worked in relative isolation (in their own silos) are mandated to work together. Another feature of this legislation is it requires states to develop this plan for all public roads in the state; not just the state owned roads. Most importantly the SHSP process directs the states to identify the safety problems and issues with crash and other relevant data. The term typically used is that problem identification is data driven. These problems, identified through the data analysis process, can then be addressed with the multi-disciplinary approach represented when the various safety partners and agencies work together. Since crashes usually have multiple causes, frequently cutting across the driver, infrastructure, and vehicle, this multi-disciplinary approach is imperative to reduce the numbers of crashes.

RSS2007 overviews the state-of-the-art research and technologies concerning key themes in safety and simulation, such as roadway design, emerging technologies and human factors. What the expected outcomes and impacts of this interdisciplinary conference could be?

For decades, the aerospace industry has applied interactive simulation technologies both as part of aircraft/cockpit design, and for the formal training of its crews…and to considerable success. In more recent times, these traditionally expensive systems have become available to more and more industries. With the exception of young driver and commercial driver training, however, the roadway infrastructure industry has yet to fully leverage the value that driving simulation technologies can provide to improving safety. Simulation of the highway driving situation is far more complex than aviation situations. It is only in recent years that the computer power and software sophistication allows this complexity to be simulated. This conference should highlight recent advances in portraying the driving situation so the effectiveness of simulation scenarios continually improves. Simulation and visualization use in the design process is a relatively recent development. Hopefully this conference will highlight simulation techniques, efficiencies and lower costs which will make use of simulation as part of the highway design process more effective and appealing.
Filippo Novelli’s Interview

On the fourth anniversary of ATS Review’s first issue we have interviewed Filippo Novelli, the artist who has been assigned to illustrate the front cover.

Filippo Novelli was born in Rome in 1969. He has been cultivating his passion for art since he was very young refining his studies at the San Giacomo Institute in Rome and then took the diploma of master in the fine Arts at the Venturi Institute in Modena. We have met him in his studio while painting the cover of the next issue of ATS. The theme is Rome, a traffic-lights over a cross-roads in EUR district.

How was born the idea to cooperate with the ATS?

I believe that nowadays artists are called more than ever to develop a poieutic art, that is an art based on research, exploration and creation. The primary purpose of my art is to combine art and science as a result of innovation in a new tech-mind and syncretic renaissance that is being developed not only in the artistic laboratories but also in the scientific ones. While attending the Faculty of Engineering a professor of mine taught me that a real engineer is the one who tries hard to apply science to the real world. Likewise a demiurge melting science to the technique; so the artist must strive every day of his life to link Nature to the intangible world. He is meant to forward a comprehensible message. The art as a means to raise the Spirit, the science as a means to raise the human civilization. When I met the Editors of ATS in 2003, I immediately thought that could be a good opportunity to realize this project. They first asked to me to make the logo for the review. At that time it had not been considered an assignation for the drawing of the cover. I drew the sketch of the logo and I sent it by e-mail to the Editor. The day after I found it reproduced on the first number of the review. It worked! As the review sketched out the first steps in the field of research, so the cover which was born as just a sketch was now developing the themes dear to ATS.

What about the subjects and the artistic research that is behind the works presented on the covers?

They show some characteristics of the Italian artistic tradition such as Modigliani’s linearism, De Chirico’s Surrealism and Metaphysical with his Dummies, the Boccioni’s Futurism with the exaltation of speed. This is my artistic background, the starting point from where I developed my theory of fluid dynamic. Lines of energy, of trajectories, of colourful echoes that spread out from the bodies. The objects project in the environment. In this sense the flow of vehicles runs through the belt of the dummy (Vol. I) and again the running man who projects his image in the space (Vol. X), the trees sucked by the speed of the vehicle driving on the road (Vol.VII).

And what about the themes you are going to develop in the next issue?

Till now I have been drawing subjects regarding circuits, laboratories, crash test dummies in the ordinary issues, whilst representing real places, through the drawing of well-known infrastructures. Now I am slowly trying to reverse this process in order to forward a message much closer to the public. My last work shows, in fact, a cross-roads in Rome, what better occasion to commemorate the RSS2007 Capitoline Conference?
From November 2003, 13 ordinary issues and 3 special issues have been published.

More than 110 referred papers have been published.

The published papers come from about 40 different countries all over the world.