HIGHWAY CAPACITY MANUALS FOR ASIAN CONDITIONS

Karl-L. Bang, Phd
President
Transportation Research and Engineering Ltd (TRE)
Jl. Pada Betah 3/170D
Bandung 40143 Indonesia
Fax/tel +62-22-211595

abstract: Highway capacity manuals (HCM) from developed countries may lead to serious prediction errors of traffic performance if applied in developing Asian countries due to major differences in driver behavior, traffic composition and level of roadside activities. Manuals for urban and rural road traffic facilities are therefore currently being developed in Indonesia and Malaysia, and similar work will soon start in China.

The Asian calculation models mainly differ from western manuals regarding
- discharge rate through uncontrolled conflict points in intersections;
- free-flow speed on road links;
- impact on speed-flow relationships for road links by roadside activities.

1. BACKGROUND

In the rapidly developing, densely populated countries in Asia considerable resources are invested in road transport which is seen as a sector which is crucial to the development effort. In designing new roads and when maintaining and upgrading existing ones, procedures are needed for the estimation of traffic performance if best use is to be made of the resources spent for construction and maintenance. Two different tools have been developed internationally over the years to meet these demands:

a) Highway capacity manuals (HCM), which emanate from the traffic engineering profession and are used for prediction of traffic performance measures (speed, delay etc) as a function of traffic interaction, geometric design and traffic control features.

b) Highway design and maintenance models (HDM), which originate from the highway engineering profession and are primarily used for selection of pavement management strategy by means of comparisons of road user costs and highway costs for different pavement types and treatments. Free-flow speed is predicted as a basis for these calculations, which normally doe not include "congestion effects".

This paper concentrates on the development of manuals of the first category (HCM). Studies at the Institute of Technology in Bandung, Indonesia (ITB) in the eighties confirmed that capacity manuals from developed countries (US, Sweden, Australia) produced misleading results due to the very different traffic composition, driver behavior and side friction caused by roadside activities in Indonesia. An Indonesian HCM project (IHCM) was therefore started in 1990 by the Directorate General of Highways within the Ministry of Public Works. A similar project (Traffic Study Malaysia, TSM) was started 1994 in Malaysia by the Highway Planning Unit within the Public Works Ministry. The Ministry of Communications in China will also start a Highway Capacity Study (CHCS) in late 1995 as a part of the China National Highway Project (Hebei - Henan). The author is engaged as Team Leader or Advisor in all these projects.

In the first two project phases of the IHCM project (1991-1994) capacity manuals were developed for urban and for interurban road traffic facilities. The IHCM project is now in its third and final phase (1995-1996), which includes development of computer software, traffic engineering guidelines and dissemination of the final results. Since the Malaysian HCM project so far only has produced preliminary results, the bulk of this paper is based on the Indonesian experience.

2. GENERAL RESEARCH METHODOLOGY

All the above-mentioned projects are partly **research projects** aiming at expanding the knowledge about Asian traffic characteristics, partly **applied studies** implementing the research results in the form of Highway Capacity Manuals for daily engineering use in each country. The main hypothesis behind the research part is that traffic characteristics in the studied Asian countries are significantly different from those in western nations. The aim of the experimental research work in each project is therefore to explore fundamental road traffic characteristics by means of field data collection and analysis.

The general strategy for the **data collection** is to register and store the type, time-space trajectory and behavior of each individual vehicle. This is performed with a combination of automatic roadside detection devices and manual observations using video camcorders. During the **data reduction** phase, information is extracted from these videos in project traffic laboratories, and added to the automatically detected event files. Aggregation of the data from individual vehicles to averages for time intervals, sites and whole populations is then performed in successive steps and stored in site condition and traffic condition data bases for statistical analysis.

Mathematical modelling is then applied to explore the relationships between dependent traffic performance variables and significant independent variables. Two different types of mathematical modelling are used, Explanatory modelling and Empirical modelling.

Explanatory modelling requires that the researcher has an idea of how driver decisions are affected by key factors and parameters. As a typical traffic engineering example, the capacity of a minor road approach in an unsignalized intersection with a major road can be assumed to be affected by the availability of gaps in the major road traffic, and the likelihood (probability function) that drivers waiting first in line in the minor road queue will accept a gap of a certain size.

Empirical modelling statistical analysis is used to determine the relevant relationships. Normally multiple regression techniques are used to develop and test the fit of a number of pre-defined mathematical functions. The models can then be validated against other data describing conditions outside the range of the original study.

In order to determine relationships between site conditions and traffic performance measures from empirical traffic data, it is necessary to conduct /data collection at a very large number of sites. An alternative way is to use a **simulation model** calibrated for actual driver behavior and vehicle characteristics. A combination of these two methods are applied in all the Asian HCM projects. Empirical data is used for calibration and validation of the simulation model (Brodin and Carlsson 1986), and for analysis of effects (e.g. environmental conditions) which cannot be studied with the model.

The VTI simulation model was originally developed by the Swedish Road and Traffic Research Institute, and is "event-controlled" and programmed in SIMULA. It describes traffic operation on a two-way single carriageway road at a micro-scopic level, in which the movements of individual vehicles are modelled as they "progress" along a defined road segment in the computer, see Figure 1. Each movement is determined from a set of stochastic attributes defining the driver behavior and the vehicle characteristics. The position and current speed of each vehicle is calculated on the basis of driver decisions, which in turn are related to external influencing factors such as road alignment and interference with other vehicles in the own or the opposing stream.

3. ANALYSIS OF URBAN INTERSECTIONS

3.1 Field data collection

The data collection for the Indonesian urban HCM was carried out in 1991-1992 at 52 signalized and 33 unsignalized intersections in 16 cities using high mounted video camcorders synchronized with a 32 channel data logger for data capture in the field as well as event recording from video playbacks in the project traffic laboratory. The collected data included traffic flow and composition, saturation flow measured as queue discharge at the stopline during green time, queue length, travel time, and driver behavior in the intersection conflict points. Geometric layout and side friction events (described by level of activity on shoulders, number of crossing pedestrians and number of exits/entries to roadside premises) were recorded manually. Figure 2 illustrates the setup for video observation of driver behavior at signalized intersections.

The vehicle behavior at the conflict point was classified in the following categories:

- U. Passes unimpeded by other vehicles.
- P. Passes without waiting for a gap, thus forcing the opposing flow to brake/stop, e.g. by pushing through the opposing traffic step-by-step.
- G. Stops before the conflict point and waits for a gap in the opposing flow.

The field data results showed that the traffic behavior at unsignalized conflicts rarely could be described as a gap acceptance process. Less than 40% of the minor road vehicles waited for gaps in the major road flow, those who did had very short critical gaps (2.1 sec).

For roundabouts the lack of an "inside priority rule"in Indonesia made the risk for congestion through blockage very high.

3.2 Passenger car equivalents

Since the traffic composition varies very much between different locations and type of traffic facility, it is essential for analysis purposes to be able to convert a mixed traffic flow into an equivalent flow of a uniform vehicle category. This is done by means of passenger car equivalents (pce) which are determined by analysis of the collected data. For signalized approaches pce are mostly determined using the Capacity Method. The basic assumption in this method is that for saturated conditions, the total discharge expressed in pcu during a given period of green time is constant from one signal cycle to another. By comparison of samples representing a large number of cycles, pce can be determined by least squares fit using multiple regression techniques:

$$Q_{g1} = LV_1 \times k_{LV} + HV_1 \times k_{HV} + MC_1 \times k_{MC} + UM_1 \times k_{UM} \qquad (1)$$
$$Q_{g2} = LV_2 \times k_{LV} + HV_2 \times k_{HV} + MC_2 \times k_{MC} + UM_2 \times k_{UM}$$
$$\cdots\cdots\cdots\cdots$$
$$Q_{gm} = LV_m \times k_{LV} + HV_m \times k_{HV} + MC_m \times k_{MC} + UM_m \times k_{UM}$$

where:
g_1 = green in cycle no 1
Q_{g1} = discharge during green in cycle no 1 (pcu)
LV_1 = number of light vehicles discharged in cycle 1 etc..
k_{LV}, k_{HV}, k_{MC}, k_{UM} is the pce for each vehicle type

By assuming that pce for passenger cars and other light vehicles (LV) is equal to 1, each equation can be expressed as:

$$LV_j = C - HV_j \times k_{HV} - MC_j \times k_{MC} - UM_j \times k_{UM} \quad \text{for } j = 1, \ldots m, \qquad (2)$$

where m = number of cycles

Based on such analysis the following pce were obtained: Heavy vehicles 1.3; Motorcycles (MC) 0.2-0.4, Unmotorized vehicles (UM) 0.5-1.0. The higher values for MC and UM refer to opposed conflicts in traffic signals and for unsignalized intersections.

3.3 Capacity of unsignalized intersections

Due to the lack of adherence to right-of-way rules by the Indonesian drivers an empirical research approach including step-wise multiple regression and normalization of the data from each individual site to pre-defined intersection layouts was used. The basic idea was to try to adjust the data for the influence by significant variables through an iterative normalization procedure. The impact on capacity of different geometric, traffic and environmental variables was then analyzed and the results regarding effect on capacity (C_i) and/or delay (D_i) was compared. A base value was chosen for the "best" variable. Capacities for individual sites and delay data were then normalized for the impact of this variable.

The analysis showed that the impact of intersection type, entry width (EW), left turning-% (LT), city size (CS) and major road median type (M) on capacity was significant for 4-way intersections. The impact of right-turning-% (RT) and split-% (SP) was less clear. The resulting calculation model for capacity C (pcu/h) is given in Equation 3 below.

$$C = C_0 * F_{EW} * F_M * F_{CS} * F_{RF} * F_{LT} * F_{RT} * F_{SP} \qquad (3)$$

The capacity for a four-arm intersection typically varies between 2,900 - 3,600 pcu/h, with a prediction error mostly smaller than 10%. This discharge rate is however very unstable and the risk for total blockage of the intersection large due to the mainly "pushing" behavior of the drivers. The resulting throughput in terms of capacity was however still impressive, with a total capacity of a four-arm intersection between two-way, two-lane roads in the order of 2900 pcu/h at an average delay of 15 sec/veh. This value is typically 50% higher than what is shown in western manuals, and higher than what can be obtained with traffic signal control of the intersection.

3.4 Capacity of signalized intersections

The capacity (C) of an approach to a signalized intersections can be expressed as a function of saturation flow (S), effective green time (g) and cycle time (c):

$$C = S \times g/c \ \ (\text{pcu/h}) \tag{4}$$

Saturation flow (S) is normally expressed as a product between a base saturation flow (S_o) for a set of pre-defined (ideal) conditions, and correction factors (F) for deviation of the actual conditions from the ideal conditions.

$$S = S_o \times F_1 \times F_2 \times F_3 \times \dots \times F_N \tag{5}$$

An iterative procedure was used in IHCM to determine the influence of nine independent variables including multiple regression to estimate F for the strongest influencing variable, and normalization for the influence of this factor through adjustment of the observed saturation flow values. Figure 3 shows the results regarding the impact of approach width on saturation flow.

For protected signalized approaches the saturation flow corresponded well with observations from western countries (S = 600 pcu/h,m). For approaches with opposing conflict the discharge of traffic was found to be heavily influenced by the poor respect for the right-of-way rule from the left. The discharge through the conflict point could be characterized as a succession of "bunches" from the conflicting streams, each obtaining right-of-way by gradually pushing forward until the lead vehicle succeeded to block the path of the conflicting stream. An explanatory model based on this observed driver behavior was developed and implemented in the Indonesian manual (Bang et al 1993). Generally it results in lower capacity than corresponding western models.

4. ANALYSIS OF ROAD LINKS

In the Asian HCM studies speed-flow relationships for road links are obtained from a combination of empirical speed-flow analysis and simulation using the VTI model for two-lane, two-way undivided roads (2/2 UD), see Section 2 above. The empirical field data is mainly used for calibration and validation of the simulation model, and for analysis of the effect on speed and capacity of cross section and environmental conditions. The simulation model is mainly used for determination of passenger equivalents (pce) and speed-flow relationships for flat, rolling and hilly terrain, where it is more difficult to obtain the necessary amount of field data.

4.1 Field data collection

Field data collection was carried in Indonesia out during 1991-1994 at a total of 150 sites, including 35 road segments with continuous residential and/or commercial roadside development which were surveyed during the urban phase of the project. In Malaysia 50 road segments have been surveyed in 1995, and the study in China will include around 100 sites to be surveyed in 1996.

The basic survey equipment at each site includes one or more short-base measurement stations equipped with pairs of pneumatic tubes (spacing 3m) connected to data loggers for recording of vehicle axle passage times. By means of specialized software traffic flow and composition, space-mean speed and headways are obtained automatically and cross-checked with the backup video recordings. Through video data reduction and identification of vehicles passing successive short-base stations (visual matching or license plate number recording), travel time and frequency of overtaking are also obtained for longer road sections in different terrain types.

Overtaking surveys using continuous, stationary video recording of 1 - 2 km long segments of roads in combination with overtaking observations from a moving observer vehicle equipped with cameras pointing forwards and backwards have also been made in Indonesia.

The following vehicle classes are distinguished in the data collection:

LV Light vehicles: Passenger cars, jeeps, mini-buses, pickups, micro-trucks.
MHV Medium heavy vehicles: Two-axle trucks with double wheels on the rear axle, buses shorter than 8m.
LT Large trucks (three-axle) and Truck combinations (truck + full trailer, articulated vehicles).
LB Large buses: Buses longer than 8 m.
MC Motorcycles
UM Unmotorized vehicles

4.2 Calibration and validation of the simulation model

The following calibrations of the VTI simulation model are performed based on the field studies:

- Base free-flow speed for ideal road and environmental conditions.
- Free flow speed at different site conditions.
- Speed in horizontal curves.
- Distribution of used driving power (determines the ability to retain the speed of a vehicle in an upgrade).
- Mean time headway between vehicles in a platoon.
- Overtaking behavior.

The calibrated model is then validated using observed journey speed, overtaking ratio and degree of bunching data from specially designated long-base sites ranging from 3 to 7 km. The field data is compared with corresponding data from simulation runs with the same road and traffic characteristics. Few differences above 3 km/h between observed and simulated speed were obtained in IHCM, the average difference was around 1 km/h.

Similar good correspondence between observed and simulated data was obtained regarding overtaking ratios and degree of bunching (leading headway < 5 sec).

4.3 Determination of passenger car equivalents

The primary methodology used for determination of pce in the Asian HCM projects is by means of simulation using the calibrated VTI model. The journey time for light vehicles (TT_{LV}) is observed as the dependent variable and calculated for each subsection of the roads and for their total length. The pce for each vehicle type is calculated as follows:

.1 Define the journey-time-flow constant for light vehicles LV at 100 % LV in the traffic flow as shown in Equation 6 where V_{600} and V_{1200} are the space mean speeds

$$\Delta T_{LV} = \left(\frac{1}{V_{1200}} - \frac{1}{V_{600}}\right) \bullet 3600/600 \qquad (6)$$

for LV at traffic flow 600 veh/h and 1200 veh/h with 100% LV. The flow levels represent a normal traffic flow range for interurban roads. The calculation is done in journey-time since the determination of speed-based pce is made regarding the effect of different traffic compositions on space-mean speed.

.2 Define in the same way the journey-time-flow constant for LV at the proportion p of LV and the proportion 1-p of type X as

$$\Delta T_{p,X} = \left(\frac{1}{V_{1200}} - \frac{1}{V_{600}}\right) \bullet 3600/600 \qquad (7)$$

where V_{600} and V_{1200} now are the space mean speeds (km/h) for LV at traffic flows with the proportion p of LV.

.3 The pce for vehicle type X can now be calculated as

$$p\Delta T_{LV} + (1 - p)\alpha\Delta T_{LV} = \Delta T_{p,X} \qquad (8)$$

where α is the pce.

Additional analysis using 5 minute speed-flow data from selected sites in flat terrain is also performed. The assumption underlying this regression analysis is that the speed-flow relationship is linear, and that pce therefore can be determined from least square fits of speed-flow samples with different traffic composition:

$$V_{LV} = A - K_{LV} \bullet Q_{LV} - K_{MHV} \bullet Q_{MHV} - \ldots - K_{MC} \bullet Q_{MC} \qquad (9)$$

Where

V_{LV}	=	Speed (km/h)
A	=	Constant representing free-flow speed
Q	=	Traffic flow for each vehicle type (veh/5 min)
LV, MHV etc	=	Light Vehicles, Medium Heavy Vehicles etc...

K_{LV} etc = Coefficient representing the speed reduction effect caused by a specific vehicle type.

The pce are obtained as the ratio between the K-coefficient for a specific vehicle type and for light vehicles. Table 1 illustrates resulting pce values for Indonesian two-lane, two-way undivided roads (2/2 UD).

4.4 Speed-flow analysis and modelling

The single-regime model (Easa and May 1980) can be calibrated to model speed-flow relationships on most road types:

$$V = FV \times [1 - (D/D_j)^{(\ell-1)}]^{1/(1-m)}; \qquad D_0/D_j = [(1-m)/(\ell-m)]^{1/(\ell-1)} \tag{10}$$

where:

- D = Density (pcu/km) (calculated as Q/V)
- D_j = Density at completely "jammed" road
- D_0 = Density at capacity
- ℓ, m = Constants

For two-lane undivided roads the speed-flow relationship is often close to linear. Figure 4 illustrates Indonesian field data from a number of sites normalized for a standard two-lane, two-way undivided (2/2 UD) urban road with carriageway 7 m. The observations are divided into two categories representing stable (marked with circles) and unstable (marked with stars) flow conditions, where the latter are defined by having density higher than D_0.

Figure 5 illustrates a schematical speed-flow model for 2/2 UD roads based on these observations. The speed at zero flow (point A) represents the free-flow speed (FV) as determined by existing conditions. If there is no speed limit (or no enforcement of existing limit) the speed drops continuously as the flow increases. An almost flat portion of the speed-flow relationship at low flow (represented by the dashed line B-C in the figure) can be observed in cases when the speed resulting from an enforced speed-limit is lower than the free-flow speed as determined by geometric and environmental conditions only.

When Q increases to a value (Q_2) close to capacity C at point D in the graph flow conditions change from "laminar" to "turbulent" with frequent speed changes. This results in a steeper speed drop until capacity (C) is reached at point E at the speed V_{cap}. When the traffic demand is near to or higher than capacity, the density will continue to increase which results in congested stop-and-go conditions with reduced flow and speed which stabilizes at V_{jam}.

Prediction of actual speed thus requires the following steps:

1. Prediction of free-flow speed FV;
2. Prediction of capacity C;
3. Conversion of the traffic flow into passenger car units;
4. Calculation of actual speed using the calibrated speed-flow model.

In order to evaluate the effect on **free-flow speed** of different site conditions, regression analysis is performed with travel time (TT) as dependent variable. The basic equation for prediction of FV developed in IHCM is as follows:

$$FV = (FV_0 + FV_W) \times FFV_{SF} \times FFV_{RC} \tag{11}$$

where:

FV	=	Free-flow speed for light vehicles at actual conditions (km/h)
FV_0	=	Base free-flow speed for light vehicles at pre-determined standard (ideal) conditions (km/h)
FV_W	=	Adjustment for effective carriageway width (km/h)
FFV_{SF}	=	Adjustment factor for side friction conditions
FFV_{RC}	=	Adjustment factor for road function class

Base free-flow speeds for different road and vehicle types obtained for Indonesian conditions for flat terrain vary between 81 - 58 km/h, for hilly terrain between 60 - 38 km/h. This is approximately 20 km/h lower than what is normal in developed countries. Between 8-10 km/h higher free-flow speeds were obtained for Malaysian roads with speed limit 90 km/h.

The basic equation for determination of **capacity** is as follows:

$$C = C_O \times FC_W \times FC_{SP} \times FC_{SF} \tag{12}$$

where:

C	=	actual capacity (pcu/h)
C_O	=	base (ideal) capacity for predefined (ideal) conditions (pcu/h)
FC_W	=	road width adjustment factor
FC_{SP}	=	directional split adjustment factor for undivided roads
FC_{SF}	=	side friction and shoulder width adjustment factor

Base capacity values for different road and terrain types obtained for Indonesian conditions are summarized in Table 2.

The VTI simulation model is used to produce **speed-flow relationships** for two-lane, two-way roads in different terrain types. The results are plotted as speed-flow diagrams, with the flow presented in pcu/h. Figure 6 illustrates presents the speed for light vehicles as a function of the flow for a flat road with 50/50 directional split. Empirical speed-flow observations for other Indonesian roads with the same general characteristics are also shown in the Figure, and seem to indicate a more linear relationship. This is also the case for Malaysian 2/2 UD roads as shown in Figure 7.

Indonesian drivers tend to overtake at short sight-distances, which reduces the slope of the speed-flow curve. The capacity for 2/2 UD is slightly higher in Indonesia than in western countries.

5. IMPACT OF SIDE FRICTION

In densely populated Asian countries there is often a great deal of activity at the edge of the road, both on the carriageway and on shoulders and sidewalks, which interacts with the flow of traffic, causing it to be more turbulent and adversely affecting performance as well as capacity.

Side friction may be taken account of indirectly, for example in the U.S. Highway Capacity Manual (TRB 1995), in which adjustment factors for capacity and service flow of multilane highways are specified according to whether a highway segment is "rural" or "suburban". The "suburban" classification is intended to reflect the greater density of roadside development and the frequency of minor junctions and driveways on suburban as compared with rural roads. The greater extent and intensity of frictional activities in Asia however requires a more direct approach to reflect the importance of side friction in capacity and traffic performance analysis.

5.1 Field data collection and analysis of side friction events

In the Asian HCM studies side friction events are registered manually on a continuous basis at the site, or recorded by means of video for later data reduction in the laboratory. The events which have been shown to have the greatest impact on speed-flow relationships in Indonesia are

- PED Number of pedestrians, whether walking along or crossing (ped/h/km);
- PSV Number of stopping and parking maneuvers (veh/h/km);
- EEV Number of motor vehicle entries and exits into and out of roadside properties and side roads (veh/h/km);
- SMV Flow of slow-moving vehicles (bicycles, trishaws, horsecarts, oxcarts, etc) (veh/h)

Regression analysis of the Indonesian data with space mean speed as the dependent variable was used to analyze the relative impact of the different side friction events on light vehicle free-flow speed as a basis for calculation of relative impact of each type of events. The relative weights obtained were used to calculate a weighted total of side friction events (FRIC) as exemplified for interurban roads below:

$$FRIC = PED \times 0.6 + PSV \times 0.8 + EEV \times 1.0 + SMV \times 0.4 \qquad (\ )$$

To simplify consideration of side friction in the speed-flow analysis, five side friction classes were defined and illustrated by means of typical photographs.

5.2 Impact of side friction on traffic performance

Side friction events have a negative impact on traffic performance over the whole flow range as discussed below:

a) Impact on free-flow speed

The desired speed at free-flow conditions is a function of road alignment, cross section and road type as well as environmental conditions regarding type of area and roadside activities. Side friction events such as stopping vehicles, pedestrians etc. reduce the desired speed in order for the driver to maintain a safe speed with consideration to the risk for unexpected roadway blockage and conflicts with other traffic elements which may suddenly appear. This effect is illustrated with a free-flow speed reduction from FV_0 to FV when the speed-flow curve intercept with the Y-axis moves from A_0 to A in Figure 8.

The impact of side friction in the most severe case for 2/2 UD Indonesian interurban roads causes a reduction of free-flow speed with a factor 0.76, e.g. a free-flow speed at 65 km/h at no side friction is reduced by 16 km/h to 49 km/h at very high side friction and narrow shoulders. Similar analysis for 2/2 UD urban roads showed that the free-flow speed is reduced from 44 km/h at no friction to 26 km/h at very high side friction as defined for urban conditions, i.e. by 18 km/h or a factor 0.59. For four-lane roads the observed side friction impact was slightly less, with a small further reduction also observed due to the existence of a median.

b) Impact on capacity

As the speed is reduced due to traffic interaction when the flow increases, the impact of side friction events on speed for reasons of traffic safety is gradually reduced. The side friction however reduces the capacity of the road due to factors such as 1) temporary reduction of carriageway width at parking and stopping manuevres; 2) change from un-interrupted to partially interrupted flow conditions due to crossing conflicts with pedestrians, entry of vehicles from minor roads and roadside premises etc. This effect is illustrated in Figure 6 by a reduction in capacity from C_0 to C, and a corresponding drop of the speed at capacity from $V_{0,CAP}$ to V_{CAP}. Side friction thus causes a reduction of speed over the entire flow range as well as a reduction of capacity. This is illustrated in Figure 8 by a shift of the speed-flow curve from A_0- D_0 - E_0 to A - D - E.

Speed-flow density regressions using the single-regime model described in Section 4.4 above was applied in IHCM for analysis of the impact of side friction on capacity. The capacity reduction at very high side friction is in the order of 25% for two-lane interurban roads. For urban roads the capacity reduction generally is 30% higher in each friction class.

6. CONCLUSION

The ongoing development of Highway Capacity Manuals in Indonesia, Malaysia and China have shown that the traffic performance of traffic facilities in densely populated Asian countries cannot adequately be predicted using western capacity manuals such as developed in the U.S, Australia and elsewhere. The main aspects for which the resulting calculation models differ are as follows:

Discharge through uncontrolled conflict points such as unsignalized intersections and opposed turning movements in signalized intersections. The capacity is heavily influenced by the common non-adherence to right-of-way rules. This generally results in rather high but unstable capacity performance at unsignalized intersections, and poor capacity performance of mixed signal phases in signalized intersections.

Free-flow speed on road links, which are generally much lower than on similar western roads.

Impact on the speed-flow relationship of road links by roadside activities (side friction), which can results in considerable reductions of speed as well as capacity.

FIGURES

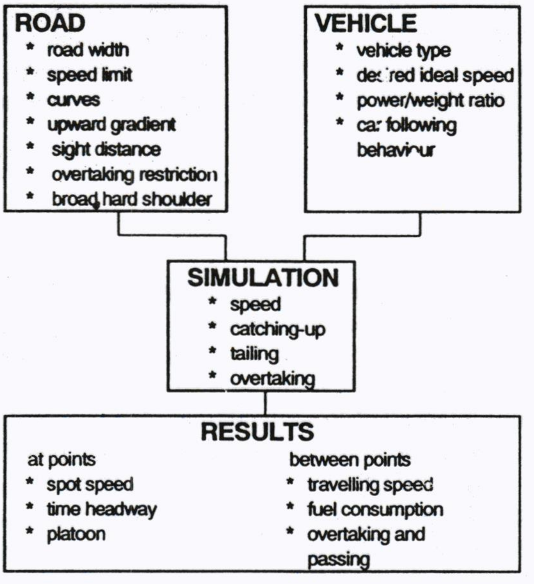


Figure 1. Overview of the VTI simulation model for two-way roads.

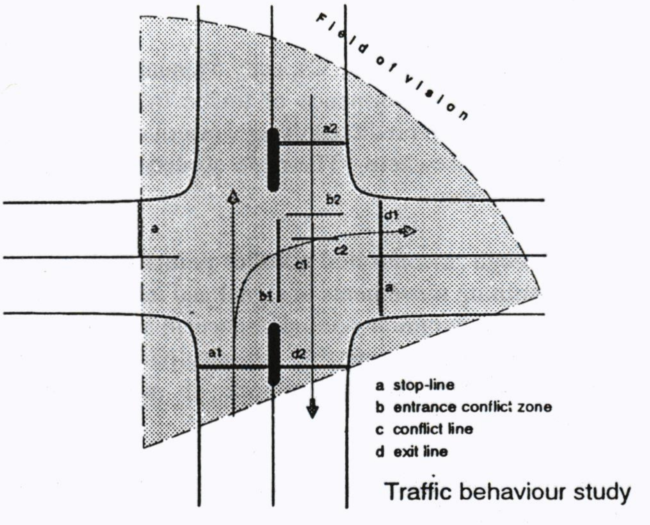


Figure 2. Video recording of driver behavior at a signalized intersection

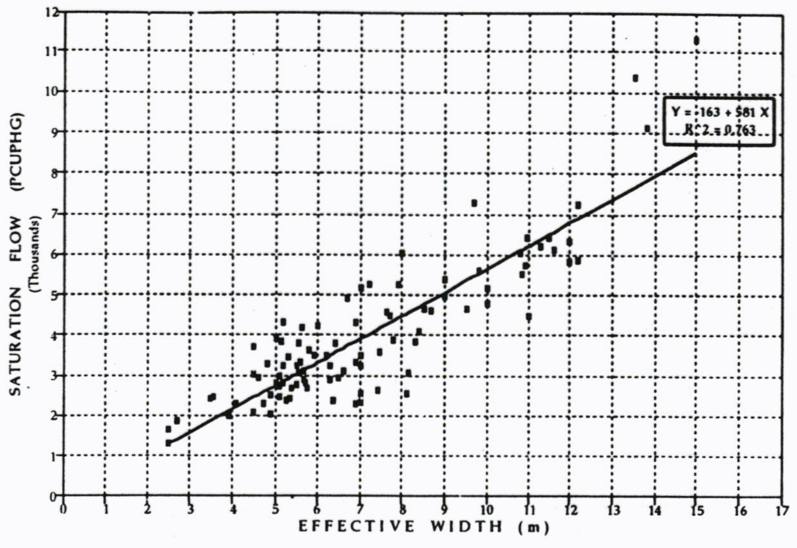


Figure 3. Saturation flow and effective width, protected approaches

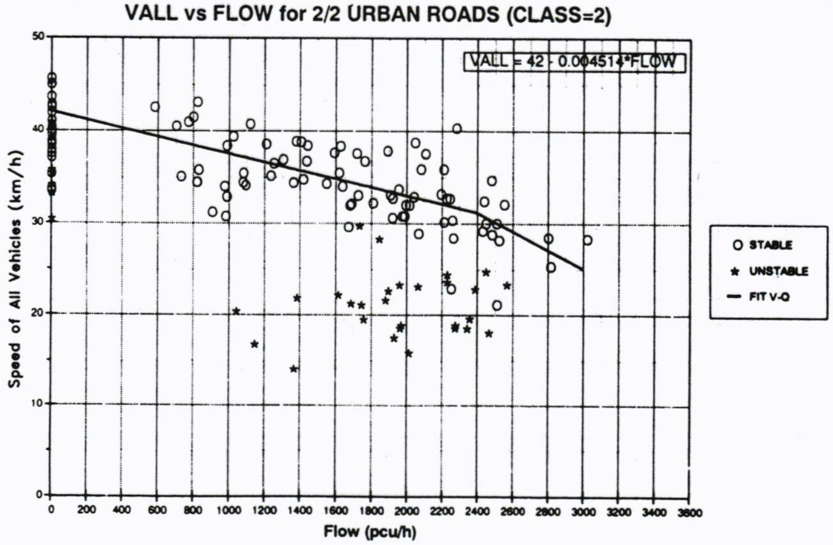


Figure 4. Field observations for 2/2 UD urban roads (IHCM)

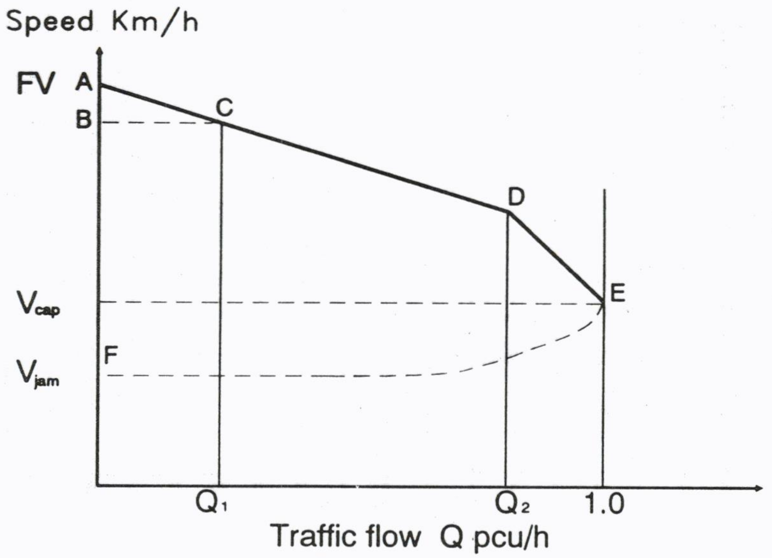


Figure 5. General speed-flow model for 2/2 UD roads (IHCM).

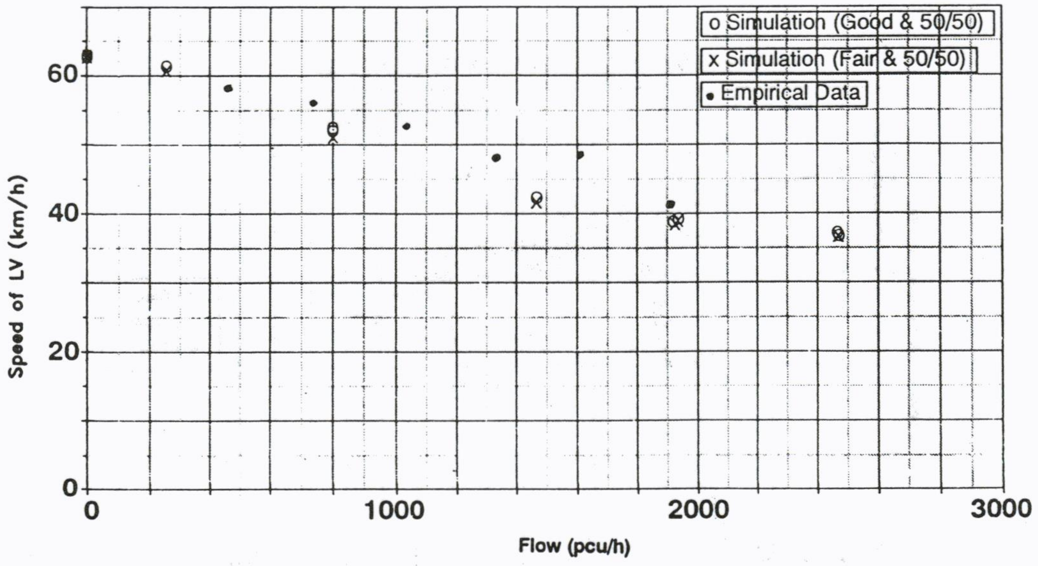


Figure 6. Speed for light vehicles (km/h) as function of the flow (pcu/h) for Flat Roads. (IHCM simulation results)

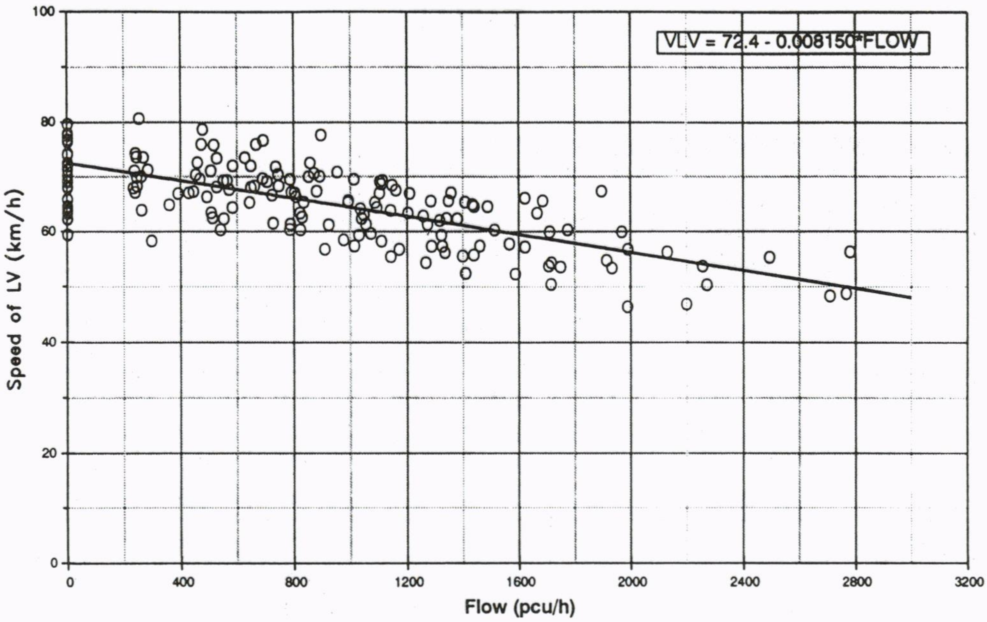
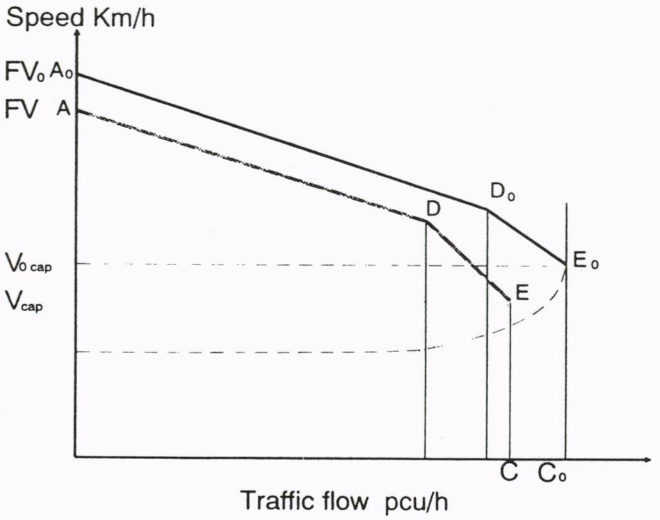


Figure 7. Speed-flow relationship for light vehicles for 2/2 UD flat roads. (Empirical data from the Malaysian HCM study).



Legend: Curve A_0- D_0 - E_0: No side friction
Curve A - D - E: High side friction

Figure 8. Impact of side friction on speed and capacity

TABLES

TWO-LANE UNDIVIDED ROADS (2/2 UD)

Terrain type	Total flow veh/h	pce					
		MHV	LB	LT+TC	MC		
					Carriageway width (m)		
					< 6m	6 - 8m	> 8m
Flat	0 - 800	1.2	1.2	1.8	0.8	0.6	0.4
	800 - 1600	1.8	1.8	2.7	1.2	0.9	0.6
	1600 - 2400	1.5	1.6	2.2	0.9	0.7	0.5
	> 2400	1.2	1.4	1.7	0.6	0.5	0.4
Rolling	0 - 800	1.8	1.6	5.2	0.7	0.5	0.3
	800 - 1600	2.4	2.5	5.0	1.0	0.8	0.5
	1600 - 2400	1.8	2.0	3.5	0.8	0.6	0.4
	> 2400	1.3	1.6	2.3	0.5	0.4	0.3
Hilly	0 - 800	3.5	2.5	6.0	0.6	0.4	0.2
	800 - 1600	3.0	3.2	5.5	0.9	0.7	0.4
	1600 - 2400	2.2	2.5	4.0	0.7	0.5	0.3
	> 2400	1.4	1.8	2.8	0.5	0.4	0.3

Table 1. Pce for 2/2 UD Indonesian roads.

Road type/ Terrain type	Base capacity (pcu/h)	Comment
Four-lane divided		Per lane
- Flat terrain	1900	
- Rolling terrain	1850	
- Hilly terrain	1800	
Four-lane undivided		Per lane
- Flat terrain	1700	
- Rolling terrain	1650	
- Hilly terrain	1600	
Two-lane undivided		Total in both directions
- Flat terrain	3100	
- Rolling terrain	3000	
- Hilly terrain	2900	

Table 2. Base capacity C_0 for Indonesian interurban roads.

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