Research Article

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Compatibility between delay functions and highway capacity manual on Iraqi highways

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Abstract: Volume delay functions (VDFs) are mathematical relationships used by the traffic allocation step of demand forecasting models to take into account the effect of increased traffic flow on the time spent to travel each possible route between different travel sources and destinations. The VDF is usually applied in static traffic assignment to describe the resultant link travel times, as a function of flow and capacity and free-flow travel time. This study aims to investigate the interface between the delay functions used by demand forecasting models and the highway capacity manual (HCM) model flow-speed relationships. The most commonly used VDFs in transport demand modeling packages in the literature were identified. The Bureau of Public Roads (BPR), conical functions (CF), Akçik and Troutbeck function (ATF), and delay logistics function (LF) were described. The four VDFs and the current HCM models were calibrated for the Iraqi road environment, and their compatibilities were examined. Results show that the best adjustments were obtained using the BPR function (quadratic error 0–0.012) and LF (quadratic error 0–0.002). The roles of CF and ATF were used with care, as both appear to neglect the delay in the condition of small to medium traffic patterns typical to country roads. Particularly, in response to single-lane roads, the LF has proven to be useful due to its potential to represent significant delays for low traffic flows and simultaneously produce more delays in congestion conditions; furthermore, the effect of flow as well as intersection spacing is obviously nonlinear. As flow reaches 600 pcu/h/lane, running time increases quickly. With more intersections per kilometer, the impact is obviously greater.

Keywords: VDFs, Bureau of public roads, conical functions, Akçelik and Troutbeck function, Iraqi roadways

1 Introduction

Roadways are a significant part of the transport system in Iraq and play a considerable role in the national economy and economic growth. The consistent level of service rendered by such services is vital to ensuring safe and cost-effective daily traffic management operations. In addition, the massive increase in the population in Iraq has contributed to an increase in demand for any part of our economy [1]. In transportation concepts, travel time is positively related to increasing the traffic flow, or with the degree of saturation. The relationship between the travel time and number of vehicles in the traffic flow is described as volume delay functions (VDFs) [2]. Thus, to describe this relationship, three components are necessary: capacity, volume, or number of vehicles and free-flow speed (FFS) [3]. FFS is defined as the desired speed that drivers feel comfortable traveling or can drive without being obstructed or influenced by other road users on the highway [4].

Delay functions, also known as VDFs, are mathematical relationships used by the traffic allocation step of demand forecasting models to take into account the effect of the increased traffic flow on the time spent to cover every possible route between different travel sources and destinations. In general, the functions included in the demand modeling packages available on the market with vehicle delays is perhaps the most critical metric used among transport practitioners to assess the efficiency of indicated crossings. The significance of vehicle latency is expressed in the application of this term in both concept and measurement activities [5].

A VDF reflects the relationship between traffic flow and travel time. As time is inversely proportional to the average speed of the traffic current, the delay functions

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are analogous to the flow-speed relations, which have been described by the traffic engineering literature since the introduction of the pioneering Greenshields model.

The Bureau of Public Roads (BPR) is a powerful function and a popular form of these functions that express the travel time in the function of the flow [6]. Newly, BPR was utilized to build a linear piece-wise approximation function, and it has been used in average-speed emission estimation models [7]. Another study used the revised BPR function to produce a better estimate of link travel times [2,8]. Akçik and Troutbeck model was proposed by Troutbeck in 1986, based on a delay model and the queuing theory method. The model was used in the SIDRA INTERSECTION 5 package [9]. The conical delay function (CF) was a new class of VDFs developed by Spiess. The parameters in CF such as steepness and capacity were used to describe the congestion behavior of a road segment [10].

Currently, the most widely used road flow-speed relationships in Iraq are those presented by the highway capacity manual (HCM), which is used by agencies and professionals in Iraq as a reference for conducting capacity studies and evaluating the quality of service on highways [11].

The HCM [12] utilizes the approximate command latency faced by automobiles at junction routes as a criterion for evaluating the quality of operation offered by the traffic signals situated at the low end of such ways. The current study aims to create compatibility of transport engineering models used for the planning and operation of Iraqi highways and to investigate the interface between the delay functions used by demand forecasting models and the HCM flow-speed relationships. To achieve these goals, the following specific objectives are as follows:

(1) To investigate the various delay functions available in the literature, for the case of homogeneous road sections of Iraq.
(2) To adjust the available relations to HCM models calibrated for highways in Baghdad, thus producing default values for the calibration parameters of a given VDF.
(3) To discuss the suitability of the adjusted models in each case.

2 Methodology

2.1 Delay functions for Iraqi homogeneous roads

The demand modeling packages available on the market incorporate a series of preprogrammed delay functions [13], which have user-adjustable calibration parameters, although default values are provided. Moreover, several of these tools also allow the user to program and use their own VDFs, which may indicate that the modeling software industry recognizes the need for local adjustments and improvements to existing models [14]. In any case, any VDF must comply with a series of requirements to be considered as a suitable function for use in-demand models.

2.1.1 Bureau of public roads delay function

The BPR delay function is probably the oldest but most widely used among the VDFs available for use. The BPR function is based on the second edition of the HCM [15] and has a parabolic format and a formulation expressed as follows:

$$t_a(v_a) = t_a(0) \times \left[1 + \alpha \times \left(\frac{v_a}{C_a}\right)^{\beta}\right], \tag{1}$$

where t_a is the travel time among two connections (min), $t_a(0)$ is the travel time of zero traffic volume (min), v_a is the vehicle flow rate of a road (pcu/h), C_a is the actual capacity of a road (pcu/h), and α and β are calibration parameters.

With $\beta > 1$, this guarantees the convex shape of the curve, which allows for increasing values of delay in demand conditions close to capacity. Historically, the default values of 0.15 and 4 have been set for α and β, respectively, although values from 2 to 12 for β have also been used in practice [16]. The higher the value of β, the shorter the delay for low traffic flows; this results in the effect of congestion becoming more delayed and sudden. The BPR function tends to underestimate the delay in the case of the congested flow when $V/C > 1$.

A recent study by refs [16,17] calibrated the role of BPR for a road in Iraq from empirical data. In the current study, a stretch of the Al-Nasiriyah city in Iraq was studied. The authors considered the α and β values of 0.21 and 3.82, respectively, as they are close to the original ones.

2.1.2 Conical delay function

With the aim of overcoming an eventual distortion of the BPR function regarding the overestimation of speed in congestion conditions, Spiess [10] developed a new VDF, named afterward as CF, which has the following formulation:

$$t(v) = t_0\left[2 - \beta + \alpha\left(1 - \frac{V}{C}\right) + \sqrt{\alpha^2\left(1 - \frac{V}{C}\right)^2 + \beta^2}\,\right], \tag{2}$$

$$\text{where } \beta = \frac{(2\alpha - 1)}{(2\alpha - 2)}.$$

In the aforementioned equation, α is a calibration parameter such that $\alpha > 1$. The development of the original conical function (CF) did not use empirical data for calibration. The objective was to seek a mathematical relationship based on basic geometry and algebra. It had a format similar to that of the BPR, but produced results capable of leading to faster and better-adjusted convergence of the traffic allocation models, especially in networks where the effect of congestion plays a central role. Spiess recommended that additional research and calibration of observed flows and speeds need to be carried out to adapt the use of this function to local conditions.

2.2 Akcelik and Troutbeck delay function

The VDF known as the Akcelik and Troutbeck delay function (ATF), which is developed for modeling traffic in congested and noncongested flow conditions, uses a model based on the queuing theory to calculate the delay in noninterrupted flow conditions [18], similar to HCM 2000 and a deterministic relationship for saturation conditions [19]. To obtain the delay factor per kilometer and starting from nonqueuing conditions in stretches of uninterrupted flow, the following equation is used:

$$u = \frac{u_0}{\left[1 + 0.25u_0(x-1) + \sqrt{(x-1)^2 + 8\tau\frac{x}{u_0 C}}\right]}, \tag{3}$$

where the parameter τ captures the delay. Lower values of τ are suggested for freeways/coordinated signal systems, while higher costs are used for arterial roads without signal coordination. In addition, u_o represents the FFS; C refers to the capacity, and x represents the degree of saturation or volume-to-capacity ratio (V/C).

2.2.1 Delay logistics function

In addition to the VDFs described earlier, some demand modeling packages [20] have incorporated a new function, based on a logistic model. The equation component used to calculate the delay in a network's spans has the following formulation as proposed by Hutchinson [21]:

$$f(x) = \frac{L}{1 + e^{-k(x - x_0)}}, \tag{4}$$

where e is the natural logarithm base; x_0 the x are the value of the sigmoid's midpoint; L is the curve's maximum value and k is the logistic growth rate or steepness of the turn.

In this section, the various delay functions described previously were compared to the HCM models adapted to the conditions of the Iraqi highways. The following conditions were considered in this analysis:
- Rural and urban highways and dual-lane highways.
- Single-lane highways.

The traffic allocation methods used by demand forecasting models consider traffic volumes as reported by the user. When multiclass allocation methods are employed, the application of equivalence factors for heavy vehicles is usually made directly to create on-demand data [22]. However, other correction factors concerning the characteristics of the road must be established in each section of the network. Later, the relationship between the V/C ratio and the delay for the types of roads studied will be shown, with adjustment factors applied as appropriate.

According to the HCM method, the equivalent traffic flow v, in pce/(h range), on highways and dual-lane highways nearby Duhok and Zakho, Iraq, as shown in Figure 1, is calculated as follows:

$$v = \frac{V}{\text{PHF} \times N \times f_{\text{VH}} \times f_p}, \tag{5}$$

where V is the total traffic volume (vehicles/h); PHF is the peak hour factor; N is the number of tracks; f_{VH} is the adjustment factor regarding the equivalence of heavy vehicles (pce/[h range]); f_p is the factor regarding the effect of the familiarity of the driver population with the road.

For the case studied, the effect of the flow on the delay per band was analyzed under standard conditions.

Figure 1: Measurement site: dual-lane highways between Duhok and Zakho, Iraq.

Thus, N, PHF, and f_p were defined as 1. In addition, as noted earlier, the conversion of heavy vehicles into matching vehicles in traffic allocation methods is usually done on demand, so that f_{VH} was defined as one, and, consequently, $v = V$ for the construction of the VDF for standard conditions.

The travel time, in turn, was calculated as the inverse of speed according to the models proposed by ref. [23] for rural equation (6) and urban equation (7) dual-lane highways:

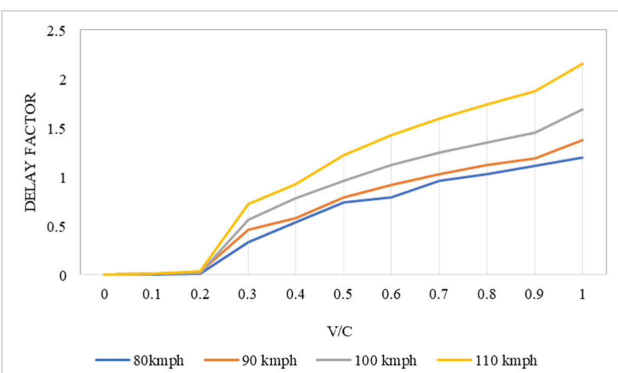
$$C = 12.5\mathrm{FFS} + 1{,}000\,, \tag{6}$$

$$C = 17\mathrm{FFS} + 1{,}000\,, \tag{7}$$

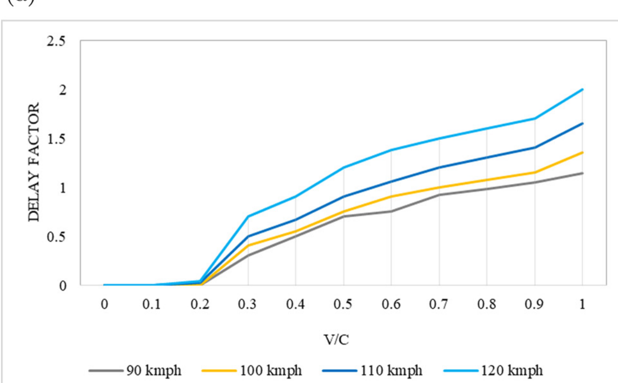
where FFS is the free-flow speed (km/h), C is the capacity (pc/h/lane).

From the analysis of equations (5)–(7), it is possible to verify that the delay varies according to the FFS value.

Figure 2 presents delay curves for rural and urban Iraqi dual-lane highways for different FFS values. Figure 2a and b show that the delay starts when V/C becomes more than 0.2. Notably, the delays are systematically more significant on urban highways, for equal amounts of V/C. Furthermore,



(a)



(b)

Figure 2: Proposed delay models for dual-lane highways: (a) rural highways and (b) urban highways.

in both cases, the lower the free-flow rate, the shorter the delay. This is because the difference between FFS and speed incapacity lies in the smaller FFS. Thus, the relative uncertainty grows with an increase in the initial FFS speed.

The calculation of the equivalent volume on single-lane highways between Nasiriyah to Al-Diwaniyah follows a formulation similar to equation (8). However, a factor related to the geometry of the f_g road is applied:

$$v = \frac{V}{(\mathrm{PHF} \times f_g \times f_{\mathrm{HV}})}\,. \tag{8}$$

The other variables are as previously defined. For generic homogeneous stretches, HCM attains f_g values for flat and undulating terrain, depending on several intervals of equivalent traffic flow [24]. Thus, adopting PHF and f_{HV} equal to 1 (base conditions), the volume of traffic to be allocated is calculated as $V = v \times f_g$. The application of this factor to the delay curves used to mark the VDFs must be adjusted. In this work, the values of f_g and average travel speed (ATS) obtained by ref. [25] were used.

To calculate the average ATS travel speed, the HCM unidirectional linear model, adapted by ref. [26], was based on the following format:

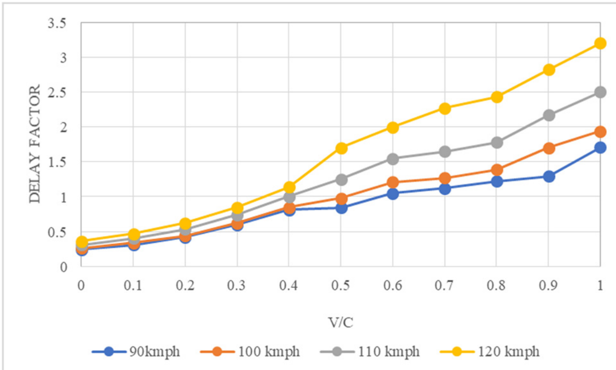
$$\mathrm{ATS} = \mathrm{FFS} + a_1 \times v_d + a_2 + v_0 - f_{\mathrm{np.ATS}}\,, \tag{9}$$

where a_1 and a_2 are calibration parameters; v_d is the traffic in the analyzed direction (pce/h); v_0 is traffic in the opposite direction (pce/h); $f_{\mathrm{np.ATS}}$ is the adjustment factor, $f(\mathrm{FFS};v_0)$, which takes into account the effect of overtaking prohibition zones (OTZ) on the average speed (kmph), established in this study following ref. [27].

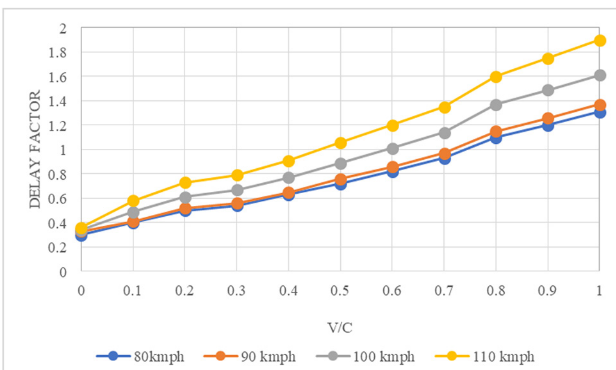
The OTZ, in turn, was defined as 20% for flat terrain and 50% for wavy terrain, as recommended by HCM.

For road sections in mountainous terrain, HCM does not provide FG values. In this case, the manual indicates the use of the factors established for specific ramps and a default OTZ percentage of 80%, as shown in Figure 3. Here, a standard division between the 50/50 senses was considered, with $v_d = v_0$.

The result shown in Figure 3 was obtained based on the conditions imposed by equations (8) and (9). The delay curves were established for different free-flow velocities and flat and undulating terrain. The relations obtained are close between types of geometry, varying slightly up to V/C of around 0.15, depending on the application of different values of f_g. Contrary to what happens in dual-lane roads, lower FFS values are linked to increasing relative delays. Although the delay factors in conditions close to capacity ($V/C = 1$) are similar to those of duplicated highways, it is noteworthy that there



(a)



(b)

Figure 3: Proposed delay functions for single-lane highways based on different FFS: (a) FFS = 110 kmph and (b) FFS = 80 kmph.

are increases in travel times even for low traffic values, which is an intrinsic characteristic of single-lane highways.

3 Results and discussion

3.1 Compatibility of existing VDFs and HCM

In this stage of the work, the delay functions presented in the previous section were adjusted to the HCM delay curves. Thus, the average delay factor was estimated for ten pce/(h range) intervals using each model to be compared.

Then, a nonlinear optimization algorithm (generalized reduced gradient) was used. The objective is to minimize the square of errors between the delay factors predicted by each of the tested models and the values obtained from the HCM models with considering the following conditions:

- For the BPR function, β must be greater than 1, as previously mentioned.
- For the CF, α must be greater than 1.
- For the ATF, the optimization method is not used. Instead, the function is adjusted using parameter x, because it is automatically related to the variables of the HCM model.

The results produced in each case are presented in terms of the calibration parameters obtained, as well as the quality of the adjustment, which are measured by adding the quadratic error of each case.

3.2 Fit for dual-lane highways

Tables 1–4 present the parameters of the VDF models described in previous sections. These were obtained for rural and urban dual-lane highways using the method described earlier. Such parameters can be used in transport demand modeling studies in the absence of a local calibration of the chosen delay function.

Comparing the errors linked to each model (Tables 1–4) for dual-lane highways, it is noted that, despite being an old model, the BPR function with quadratic error is (0.003–0.001) in flat terrain and (0–0.003) in rolling terrain, and also for logistics function (LF), the quadratic error is (0.01–0.001) in flat terrain and (0.002–0.001) in rolling terrain; therefore, the BPR function together with logistics, was the one that produced the best adjustments to the HCM model calibrated to the Iraqi highways. It is noted that, despite being an old model, the BPR function, together with logistics, produced the best adjustments to the HCM model.

Figure 4 shows this comparison for the case of a typical 110 km free-flow dual-lane highway. The reasonable adjustment of BPR and LFs stands out. Meanwhile, the CF and ATS, although initially based on the HCM's parabolic structure, have given particular focus on producing higher values of delay in conditions close to capacity and congested condition. Due to this characteristic, the solution of the least quadratic error underestimates the delay for medium or low traffic flows.

Therefore, it is recommended that the CF and ATF must be used with caution when modeling demand on rural highways, in which the relationship between traffic flow and delay in unsaturated conditions is especially relevant and congestion is less frequent than on urban arterial or collecting roads.

3.3 Fit for single-lane highways

As for the case of dual-lane highways, Tables 5–8 present the calibration parameters obtained for single-lane

Table 1: BPR function calibration parameters and quadratic error by VDF and FFS – dual-lane highways

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
α	0.27	0.22	0.15	0.08
β	2.57	2.64	2.71	2.79
Quadratic error	0.003	0.002	0.002	0.001
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
α	0.29	0.26	0.22	0.17
β	2.02	2.12	2.23	2.38
Quadratic error	0	0.002	0.002	0.003

Table 2: CF calibration parameters and quadratic error by VDF and FFS – dual-lane highways

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
α	46	96	220	513
Quadratic error	2.19	1.77	1.31	1.01
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
α	31	47	80	147
Quadratic error	2.50	2.13	1.72	1.33

highways on flat or undulating terrain. The best results of quadratic error were for BPR function (0.006–0.005) in flat terrain and (0.011–0.012) in rolling terrain, also for LF (0–0.001) in flat terrain and (0.001–0.002) in rolling terrain, while the analytical results (Figure 4) show the same analysis for various standard slopes with a length of 1 km. Again, BPR's functions and logistics produced the best fit, especially the last one. Figure 5 shows the VDFs adjusted for a typical single track road of FFS = 80 kmph in undulating terrain. Both in the case shown and for a similar

Table 3: ATF calibration parameters and quadratic error by VDF and FFS – dual-lane highways

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
x	0.135	0.098	0.058	0.021
Quadratic error	1.8	1.12	0.58	0.18
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
x	0.155	0.14	0.119	0.089
Quadratic error	2.3	1.64	1.07	0.6

Table 4: LF calibration parameters and quadratic error by VDF and FFS – dual-lane highways

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
$x1$	0.96	0.97	0.98	0.99
$x2$	1.33	1.29	1.25	1.08
$x3$	3.92	4.22	4.66	5.27
$x4$	2.45	2.56	2.69	2.84
Quadratic error	0.01	0.007	0.004	0.001
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
$x1$	0.97	0.98	0.99	0.99
$x2$	0.35	0.3	0.25	0.19
$x3$	2.91	3.19	3.55	3.99
$x4$	3.75	4.16	4.63	5.21
Quadratic error	0.002	0.002	0.002	0.001

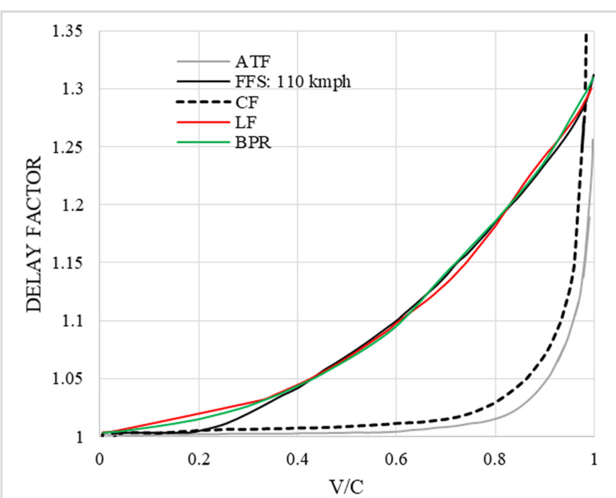


Figure 4: Comparison of delay functions for a rural double-lane highway of FFS = 110 kmph.

road on flat terrain, the best adjustment obtained through the BPR function is given using $\beta = 1.01$ (minimum value), which makes 1 linear. Notably, the use of defaults typically found in the literature ($\alpha = 0.15$ and $\beta = 4$) produces a parabolic relationship that is usually related to a duplicated highway.

Better adjustments are obtained with the use of the logistic function, as it can estimate a more marked increase in the delay for low traffic volumes, reflecting the use of the FG in the equation. In addition, the logistical function also produces more significant delays in capacity and congestion conditions, which is desirable in a VDF.

The CF, in turn, has the same deficiencies described for the case of dual-lane highways. However, such disadvantages are accentuated due to the more significant delay in conditions of low traffic flow characteristic of single-lane highways. Finally, the ATF is analogous to the HCM model itself for duplicated highways. To adapt

Table 5: BPR calibration and error parameters by VDF and FFS – single-lane highways in generic terrain

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
α	0.16	0.18	0.21	0.26
β	1.01	1.01	1.01	1.01
Quadratic error	0.006	0.006	0.005	0.005
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
α	0.16	0.18	0.21	0.26
β	1.01	1.01	1.01	1.01
Quadratic error	0.011	0.012	0.011	0.012

Table 6: CF calibration and error parameters by VDF and FFS – single-lane highways in generic terrain

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
α	28	37.9	51.2	28.3
Quadratic error	2.54	2.23	2.37	2.76
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
α	28.5	38.1	52.6	28.4
Quadratic error	2.52	2.23	2.38	2.77

Table 7: ATF calibration and error parameters by VDF and FFS – single-lane highways in generic terrain

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
x	0.9832	12	15	19
Quadratic error	6	6	5	5
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
x	0.9832	12	15	19
Quadratic error	6	6	5	5

Table 8: LF calibration and error parameters by VDF and FFS – single-lane highways in generic terrain

Parameter	Single-lane highways – flat terrain			
FFS	120 kmph	110 kmph	100 kmph	90 kmph
$x1$	0.78	0.84	0.76	0.78
$x2$	0.72	0.7	0.68	0.67
$x3$	0.75	1.12	0.57	0.66
$x4$	0.55	0.78	0.75	0.93
Quadratic error	0	0	0.001	0.001
Parameter	**Single-lane highways – rolling terrain**			
FFS	**110 kmph**	**100 kmph**	**90 kmph**	**80 kmph**
$x1$	0.66	0.89	0.77	0.77
$x2$	0.83	0.56	0.79	0.8
$x3$	0.31	1.2	0.82	0.83
$x4$	0.37	0.99	0.67	0.78
Quadratic error	0.001	0.001	0.002	0.002

it to single-lane highways, the f_p parameter in equation (5) must be set to 0. Even so, the adjustment obtained is noticeably lower than the one produced by the BPR and LFs.

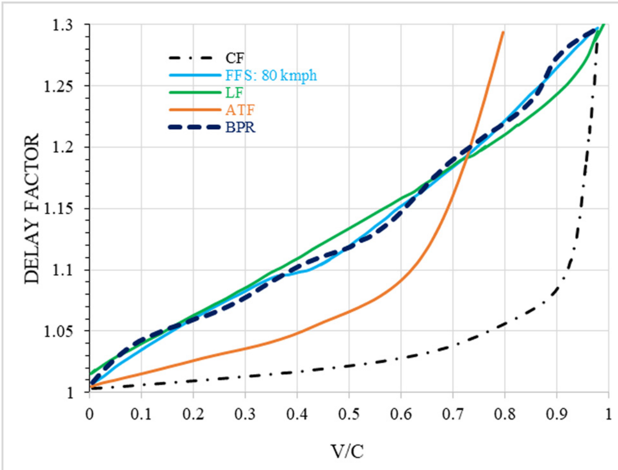


Figure 5: Comparison of delay functions for a single-track highway of FFS = 80 kmph.

3.4 Assortment of arrival headway functions

With the arrival leads from the field survey on Highway 1, Baghdad to Syria (Kameshli), three delay functions discussed earlier are examined. One statistical methodology can be used for the comparative study, i.e., a Chi-square, to determine how a quantitative distribution represents a calculated distribution. If the Chi-square (Cal) measured is smaller than the Chi-square value, there is significant evidence of similar delay functions. If the two distributions compared are statistically identical, the results will be marked "Yes"; otherwise, as shown in Table 9 (case I) and Figure 6, it will be marked "No." In the five data groups for Highway 1 from Baghdad to Syria (Kameshli), it is evident that the statistical difference between the distribution of the progression calculated and the log-normal distribution is statistically distinct, and the distribution of advancements is not statistically different from the exponential distribution negative. There are three places where there is no proof that the calculated improvement and the delay logistics functions (DLFs) vary statistically. During the nonpeak hour, another fitness test was carried out with progress results. The DLFs offers the best fit compared with the other two distributions, as shown in Table 9 (Case II) and Figure 7. The headways taken on Highway 1 (Baghdad to Syria) were used for a comparison test (Table 9, Case III and Figure 8). Both sites display an exponential or delay logistics feature similar in form.

Consequently, we find that only position 3 has the distribution equal to negative-exponential distribution and the logistics delay function. In short, the test results show that in the most appropriate routing distribution on Highway 1: The DLF is Baghdad, Syria (Kameshli). The negative-exponential distribution of Highway 1: Baghdad to Syria (Kameshli) can also be explained. The DL function is then added to Iraqi Highway 1.

Table 9: Suitability function test for the progression of delay functions on Kameshli national highway (Highway 1)

VDFs	CF			ATF			LF		
Location	Chi-square value (Cal)	Chi-square value	Outcome	Chi-square value (Cal)	Chi-square value	Outcome	Chi-square value (Cal)	Chi-square value	Outcome
Case I: Heavy Traffic									
1	2263.94	6.1	Not fitted	75.6	9.64	Not fitted	3.67	11.64	Fitted
2	1021.1	9.5	Not fitted	9.35	9.64	Fitted	5.36	11.57	Fitted
3	840.24	11.2	Not fitted	34.98	9.64	Not fitted	23.15	11.69	Not fitted
4	806.34	9.4	Not fitted	25.69	9.64	Not fitted	5.61	11.34	Fitted
5	517.1	11.4	Not fitted	30.48	9.64	Not fitted	7.2	11.94	Fitted
Case II: Less Traffic									
1	625.37	10.25	Not fitted	20.67	8.7	Not fitted	7.95	12.02	Fitted
2	1034.8	9.64	Not fitted	11.24	8.7	Not fitted	8.92	12.02	Fitted
3	940.64	10.64	Not fitted	11.64	8.7	Not fitted	11.02	12.03	Fitted
4	1102.94	11.01	Not fitted	4.64	8.7	Fitted	6.37	12.04	Fitted
5	568.94	10.94	Not fitted	2.96	8.7	Fitted	1.2	12.04	Fitted
Case III: Fitness test for highway development									
1	2761.4	7	Not fitted	125.3	14.62	Not fitted	16.94	15.64	Fitted
2	1635.8	7	Not fitted	136.4	15.04	Not fitted	20.24	15.33	Not fitted
3	795.64	7	Not fitted	162.48	14.96	Not fitted	22.34	17.52	Not fitted
4	691.24	7	Not fitted	160.47	14.35	Not fitted	30.11	19.22	Not fitted
5	1104.1	7	Not fitted	122.3	14.33	Not fitted	19.57	20.24	Not fitted

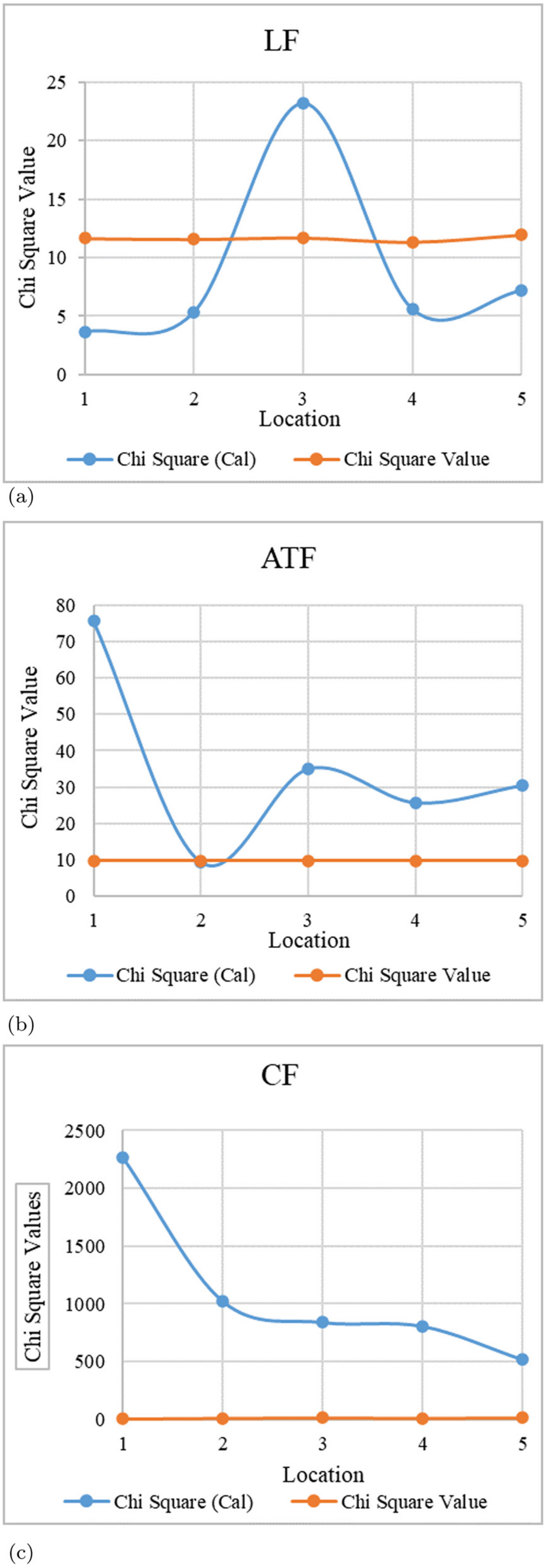


Figure 6: Suitability function test for the progression of delay functions on the Kameshli National Highway for heavy traffic, Case I: (a) LF; (b) ATF; and (c) CF.

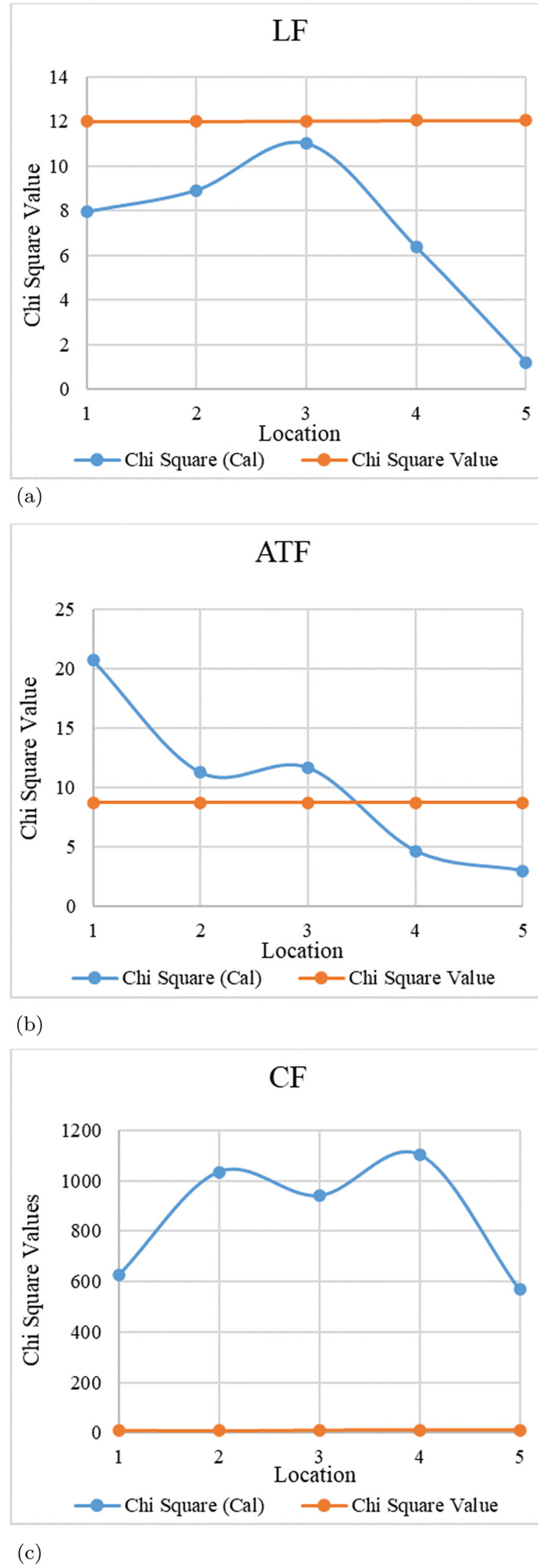


Figure 7: Suitability function test for the progression of delay functions on the Kameshli National Highway for less traffic, Case II: (a) LF; (b) ATF; and (c) CF.

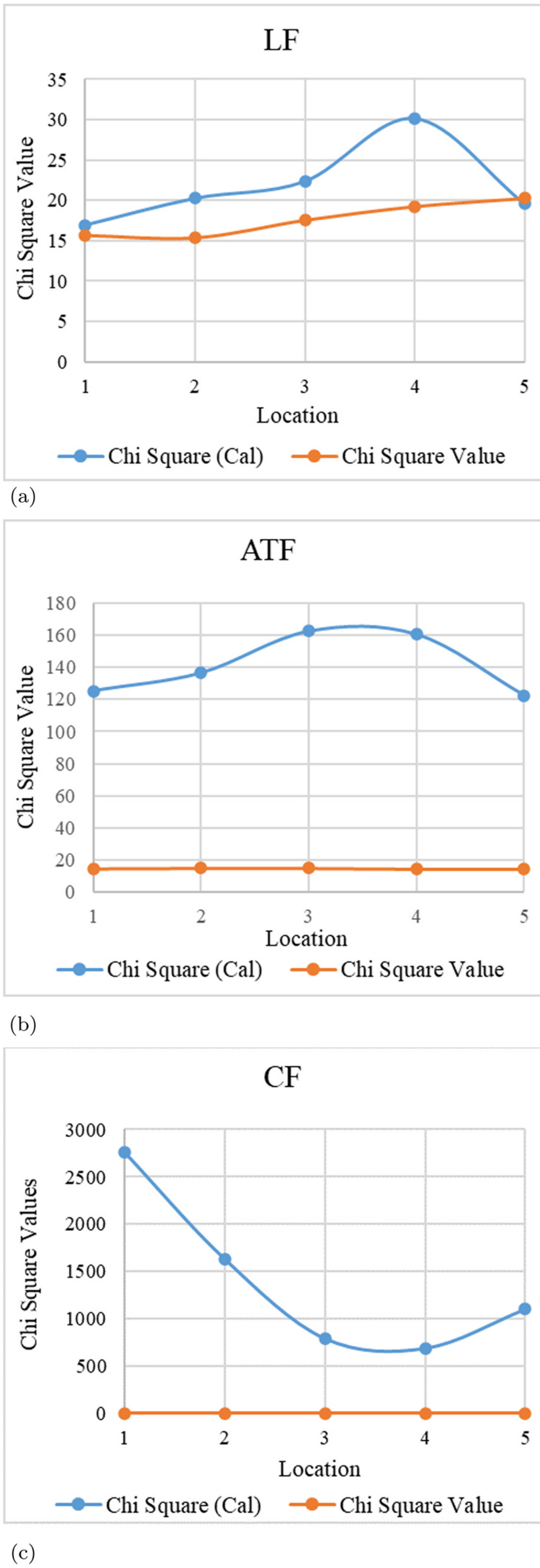


Figure 8: Suitability function test for the progression of delay functions on the Kameshli National Highway for the fitness test for highway development, Case III: (a) LF; (b) ATF; and (c) CF.

3.5 Considerations regarding the applicability and limitations of the proposed parameters

The parameters proposed in this work were obtained through the definition of an optimization problem, in which the coefficients of each tested model must produce relations as close as possible to a reference model, in this case, flow-speed relations calibrated for Iraqi highways.

Thus, the results achieved can be considered valid for road segments with characteristics similar to those used in Brazil by de Andrade and Setti [23], which consisted of highways and dual-lane highways typical of the governorates of Iraq. For different homogeneous stretches, various factors are expected. Thus, local calibration is desirable.

In relation to the above, it is necessary to have observations of the flow rate and average speed of the traffic flow for intervals of 1 h or less. A trace of the flow versus the delay must be created based on the current speed of the free-flow of the road. Finally, using these data, an optimization technique is applied, as previously mentioned, to minimize the difference between the square of errors of the chosen function and the reference model.

3.6 Factors impacting running time investigation

The HCM [12] is an approach focused on arterial segment values collected. These values are believed to be constant and are only subject to segment length, arterial class, and free-flow velocity. It is widely considered illogical to conclude that running times are not contingent on arteries and are contrary to the HCM paradigm of uninterrupted flow systems. Prassas [28], who used simulation to show that running speed changes with the flow, recently challenged this theory. The author used empirical data from the Center for Iraqi Transport Studies (CITS) to examine factors that influence the measurable runtime per kilometer. The survey consisted of single-way arterial sections with an average distance between 240 m and 935 m. The data set consisted of 570 measurements of listed 15-minute traffic volumes and calculated corresponding operating times and travel times by sample cars. The works were translated into flow rates in pcu/h using the same data calibrated for passenger car equivalents.

A multilinear regression was conducted with substantial effect to test the hypothesis that the flow rate and intersection size do not influence the running time. There are two independent variables, namely, "q" and "f," which means that the running time is not independent. At 99.99 % of confidence, all variables proven statistically based on flow can be rejected. The findings show that each additional junction raises its operating time by 6.8 seconds by an average of over 1 km in the arterial length. In contrast, each 100 pcu/h increase inflow increases by 2 seconds. However, the relationship is not inherently linear, and there is substantial unexplained variability in observed operating times, as shown by the low R2. The precise definition of the running time is a fundamental issue here. In the CITS survey, the cumulative time when a test truck moved (i.e., running time is equal to the less time-stopped time of journey) is determined. However, cycles in which vehicles travel slowly before and after stop are included in the service time [29]. This extra time is the difference between a control time delay and a stopped phase, known as the delay of deceleration–acceleration. The HCM approach means that the control delay should be omitted from the running time (1). The running time corresponding to the HCM should, therefore, be a net operating time, which in practice, is very difficult to calculate. Arguably, the "net time" is independent of the flow rate. In this work, the networking time was determined by the exclusion of 30% of the time stopped by the calculated running time to test this hypothesis. This calculation is based on the well-known assumption that the delay is around 1.3 times the delay stopped.

The findings of the second regression are dismissed at 99% confidence, and the hypothesis that the net time of operation is independent of the flow and the number of intersections per kilometer is proposed. The overall effect of the two variables is less powerful than that with the calculated time, but is still very important. The observation was aggregated into three classes of intersection distance (1–2, 2–3, and more than three signals per kilometer) and six flow range at 200 pc/h increments to examine the essence of the relationship between run time and flow. The effect of flow and intersection spacing is nonlinear. As the flow reaches 600 pcu/h/lane, running time increases quickly. With more intersections per kilometer, the impact becomes more significant.

3.7 Possible HCM model enhancements

It appears that for typical HCM method arterial parameters, an aggregate speed-flow correlation could be formed. The key issue with the new HCM system is that the traffic volume and the arterial operating time are not related. It is understandable that the future update to HCM will be based on the relationship. It should be noted that a clear description of running time should be used in this revised model. If the time of movement is calculated as the moving time of the vehicle, the deceleration–acceleration in equation (1) is risky to be double counted. This is because the time lost because of slow motion before and after a stop is often included in control delays, although theoretically part of the running time. One way to get the networking time from the time calculated is to deduct 8.6 s for each stop. That is the average value observed by the author in another analysis for the deceleration–acceleration delay. This system, however, ignores partial stops, which occur in cases where vehicles slow down without stopping. Possible improvements to the CITS model were shown to be capable of replicating travel speeds across the range of conditions encountered in Singapore. It should be optimized and checked using local data to be useful for planning applications elsewhere. In a future analysis, new factors such as the flow rate and the lane ratio (as described above) may be investigated. Although it is not helpful to use a green split ratio (as its value can shift with an increased flow), additional general parameters that represent a signal control type along the arterial can be used, including the number of phases.

The selection of the minimum intersection delay parameter is a practical problem when using the CITS model. The importance of the signal depends on signal timing and progression efficiency. A recent survey of five intersections has shown that, based on the length of the green and red cycle and the proportion of vehicles that entered the green sector, the minimum stop-delay ranged from 3 to 25 s. A low saturation value, i.e., 5%, can be used to estimate the cost for minimal delays from the typical delay formulation. The HCM formulation was delayed and projected close to observations.

4 Conclusions and recommendations

- A wide range of delay functions (VDFs) was analyzed, describing those most cited in the literature and often used in transport application modeling packages: (1) BPR function, (2) CF, (3) ATF, and (4) logistical function.
- From the analysis of the literature, it could be inferred that, most of the VDFs or default values suggested for their calibration parameters are related in some way to

the flow-speed models presented by the HCM for freeways and dual-lane highways (multilane highways) since its second edition. Thus, this work tested the compatibility between the four VDFs described and the current HCM models of the Iraqi highways.
- The main conclusions are that the best adjustments are obtained using the BPR function (quadratic error 0–0.012) and the logistic function (quadratic error 0–0.002). Specifically, compared to the traditional BPR function, the logistical role proved to be especially attractive on single-lane highways due to its ability to reflect the most severe delays for low traffic flows and simultaneously produce more delays in congestion conditions, which is a desirable feature in demand modeling.
- The findings show that each additional junction raises its operating time by an average of over 1 km in arterial length by 6.8 s, whereas each 100 pcu/h increase in flow increases by 2 s. The effect of flow as well as intersection spacing is obviously nonlinear.

5 Future recommendation

For the future work, it is recommended to adjust the VDFs for more ramp configurations specific to single-lane roads, with different values for FG. Besides, it is desirable to analyze the delay factors produced under demand overcapacity ($V/C > 1$), case by case, to verify whether the values proposed in this work for the noncongested regime (curves HCM calibrated) do not generate inconsistent results for the congested regime.

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