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Classificazione Decimale Dewey:

388.31 (23.) TRASPORTO VEICOLARE. FLUSSO E MANUTENZIONE DEL TRAFFICO

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ROAD TRAFFIC, THEORY AND TECHNOLOGY

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ISBN
979-12-218-2077-5

PRIMA EDIZIONE
ROMA 11 LUGLIO 2025

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Chapter I

The parameters of vehicular flow

1.1. The traffic flow conditions

Given that the motion of road vehicles is operated on sight, in the literature a distinction is made between roads operating in conditions of "uninterrupted flow" and "interrupted flow":

a. a road is in a state of uninterrupted flow when the motion of vehicles is not stopped for reasons external to it: i.e. the presence of elements capable of interrupting its flow (traffic lights etc.) is excluded. Any interruptions due to accidents (with partial or total reduction of the useful section), excess demand with respect to capacity at critical points (entries, exits) or propagation of disturbances (shock waves) in dense vehicular streams, being internal causes, lead to the belief that the road is still in uninterrupted flow. These flow conditions are, therefore, the result of interactions between vehicles of the same stream and between vehicles, geometry and the environment where the road is developed
b. is, instead, in a state of interrupted flow a road where vehicular stops are frequent due to elements external to it (traffic lights, STOP, pedestrian crossings etc.) that impose interruptions to the flow regardless of the volume of traffic.

The following discussion refers to the uninterrupted flow as it can be found in the road segments not influenced by points of conflict (the so-called basic road segments) which are found almost exclusively in the non urban road network.

1.2. The parameters of the vehicular flow

The main parameters used in the traffic flow to characterize the conditions in which the flow occurs are:

— the flow (q), the number of vehicles passing through a section of a lane in a given time interval
— the density (k), number of vehicles that at an instant t are contained in a given lane [*ln*] length
— the speed (u), spatial average of instantaneous speeds of vehicles that are contained in a given lane length at an instant t.

The parameters mentioned are related by the steady-state equation:

$$q = uk \quad [veh/time] \tag{1}$$

it is also

$$q = u/d \quad [veh/time] \tag{2}$$

finally

$$q = (d/u)^{-1} = 1/\tau^{(1)} \quad [veh/time] \tag{3}$$

$d = 1/k$ = spacing, distance between corresponding points of successive vehicles [*space/veh*]
$\tau = d/u$ = time interval (*headway*) between corresponding points of successive vehicles [*time/veh*].

The relationship between flow, average spatial speed and density cannot be seen only in a deterministic sense as expressed by the equations reported: in fact, vehicular flow, within limits that are not exactly definable, is characterized by random distributions of gaps and, therefore, it is also a random phenomenon. Furthermore, since both the flow and the density depend on the speed and that as the density increases the average speed of the vehicular stream decreases, it is more correct to write the equation of state in the form:

$$q(k) = u(k)k \tag{4}$$

Numerous experiences have shown the dependence of vehicular flow on speed and density in the sense that, as the latter increases, free

(1) Equation (3) does not say at what speed the flow q is made: indeed, a flow of 3600 *veh/h-ln* can be achieved with vehicles travelling on a lane with an headway τ=1^s evenly spaced out by 15 or 20 *m* at the speed of 54 or 72 *km/h*.

driving becomes increasingly conditioned with significant reductions in the average speed due to the influence of slow vehicles. Subsequently, the relationships between the aforementioned parameters will be analyzed in detail. As for the capacity, understood as the maximum number of vehicles that, under given conditions, passes through a section of lane in the unit of time, the 2010 and 2016 Hcm manuals will be followed for its definition and determination.

1.3. The vehicular flow rate

To choose the length of the period of time to which the vehicular flow refers, especially if aimed at the design of the road section, it must be considered that:

- the observation period must be sufficiently short to ensure an almost stationary flow; that is, the number of vehicles passing through the elementary intervals into which the duration of the observation can be divided must not be very variable because, in order for the average number of passes per unit of time to no longer be considered a random variable and to be substitutable by the average value, it is necessary that the relative dispersion be small
- the monitoring period must also be of such duration as to be able to represent the variability of the flow over time with a number of values that is not excessive and dispersed; for instance, if one wanted to describe the variability of the hourly flow rate in a year, 8760 values would have to be compared; if, on the other hand, one wanted to describe the variability of the flow rate of ¼ of an hour in a year, one would have to compare a quadruple number of values (35 040) which will have a higher relative dispersion than the previous one.

That said, it was considered that the hour represents a situation of compromise between the opposing needs proposed.

1.3.1. *The variability of the flow rate in an hour*

By detecting the number of vehicles passing through a road section in each of the *n* elementary time intervals into which the hour can be divided (e.g., 60 intervals of 1^{min}), the phenomenon can be represented in the form of a frequency distribution (counting distribution) that is fairly approximated to that of Poisson if the flow q it is far from capacity and, therefore, the vehicles are not mutually affected and the conditions of randomness of the flow are realized.

In this case, the Poisson distribution expresses the probability that in a given time interval (t) occur (x) vehicle passes when the average number of passes in that interval is (qt):

$$P(x) = (qt)^x e^{-qt}/x! \qquad (5)$$

Assuming that the phenomenon of the passage of x=0, 1,..., k vehicles in a given range t is a random phenomenon obedient to probabilistic laws of the type mentioned above (it is reasonable to think that the causes that determine P times the pass of x vehicles in the range t are numerous and independent as long as the vehicular densities are modest), it is observed that the relative dispersion of the data, i.e. the variability of the "no. vehs./interval", measured by the relative dispersion coefficient C_d:

$$C_d = SD/average \qquad (6)$$

SD = standard deviation

decreases as the average value increases (in the Poisson distribution C_d=1/√qt), i.e. as the reference time interval for the observation of the passes increases. Indeed, passing from an interval of 1^{min} to one of 5 or 10^{min}, the variability of the "vehs./interval" decreases because the average value increases and, therefore, in accordance with (6), also the relative dispersion coefficient.

From this it follows that the flow rate measured in a period of time $<1^h$ is more likely to exceed its average than the hourly flow rate; this property, characteristic of the distribution of Poisson and others, explains why it is appropriate to refer to the hour in traffic flow measurements. On the other hand, if the length of the reference range were $>>1^h$, it would be reckless to consider that there are conditions of stationarity of flow within this range.

By choosing the hour as a time reference, we try to achieve a condition of statistical equilibrium (probability of passes equal in time) or, in other words, that the relative dispersion of the number of vehicle passes with respect to the average is small. In this way, the number of passes will no longer be considered as a random variable because it can be identified by the average value, which has now become a deterministic parameter. Ultimately, the choice of the breadth of 1^h as the reference interval to express the flow rate of a road is a compromise between conflicting needs.

Indeed, the variability of the flow (rhythm of passes) within the hour would not constitute a problem if the road section were sized to provide a high level of operation in correspondence with the hourly rate of flow of the design: indeed, the peaks of traffic that could occur within elementary intervals of the hour would only cause a momentary decay of the operating conditions. If, however, the section were designed to provide, at the same hourly rate of flow, medium-low operating characteristics, there would be a risk of having, in peak periods, conditions of incipient congestion.

In general, during an hour of continuous vehicle survey, the traffic flow is not stationary and the hourly flow rate, understood as the average relative to the hour of the law of probability of the passages of x vehicles in its fractions, is greater than the number of vehicles actually transited. Therefore it is customary to assume as the design flow rate the average, relative to one hour, of the law of probability of the pass of x vehicles in a time interval shorter than an hour (15^{min}), also because, in this interval, the traffic flow is approximately stationary and statistically stable.

The flow rate of an interval $<1^h$ reported per hour is called "traffic intensity" (*TI*): it can be understood as the hourly flow rate that would be achieved if the flow measured in a fraction of an hour remained constant in all the intervals into which the hour can be divided.

Having a continuous flow survey extended to 24 hours, it is possible to identify the "maximum traffic intensity" (*MTI*), the product of the highest flow measured in a given interval of an hour of maximum flow over 24 hours (peak hour, *PH*) by the number (n) of such intervals contained therein.

1.3.2. *The peak hour factor*

The peak hour factor (*PHF*), a dimensionless parameter that provides an indication of the variability of the flow within an hour and, in particular, of the hour of maximum load over 24 hours, is expressed by the equation (7) where q_e [*veh/h*] is the actual hourly volume:

$$PHF = q_e/MTI \quad [\leq 1] \tag{7}$$

If *MTI* is computed on a 15^{min} basis (MTI_{15}), in theory $0{,}25 \leq PHF \leq 1$; computed on a 10^{min} basis, it is: $0{,}1\underline{6} \leq PHF \leq 1$. As a rule *PHF* assumes values between 0,80 and 0,95: the closer it is to 1, the less variable the number of vehicles per elementary interval of an hour. The *PHF* depends on the socio-economic characteristics of the area affected by the road: the presence of settlements of considerable importance or of particular territorial structures (factories, schools, stadiums etc.), can lead to significant increases in demand concentrated in short periods of time at the hours of entry/exit, causing it to assume very low values. Fig. 1 shows a flow survey in the time slot 7–9 a.m. where the maximum peak hour is located on 24^h.

It can be seen (Fig. 1) that the real hourly volume q_e is 1622 *av/h* (*av*=passenger car), while the *MTI* based 5^{min} (MTI_5) is of 2232 *av/h* and the one in 15^{min} based is of 1980 *av/h* which correspond, respectively, *PHF* of 0,73 and 0,82. The hour interval where the maximum vehicular load is obtained is called *critical interval* (*CI*).

It should also be noted that the 15^{min} intervals highlighted in the Fig. 1 are indicated sequentially in discrete form for graphic needs: in reality, the 15^{min} peak flow can be placed in any interval of 15^{min} consecutive within an hour.

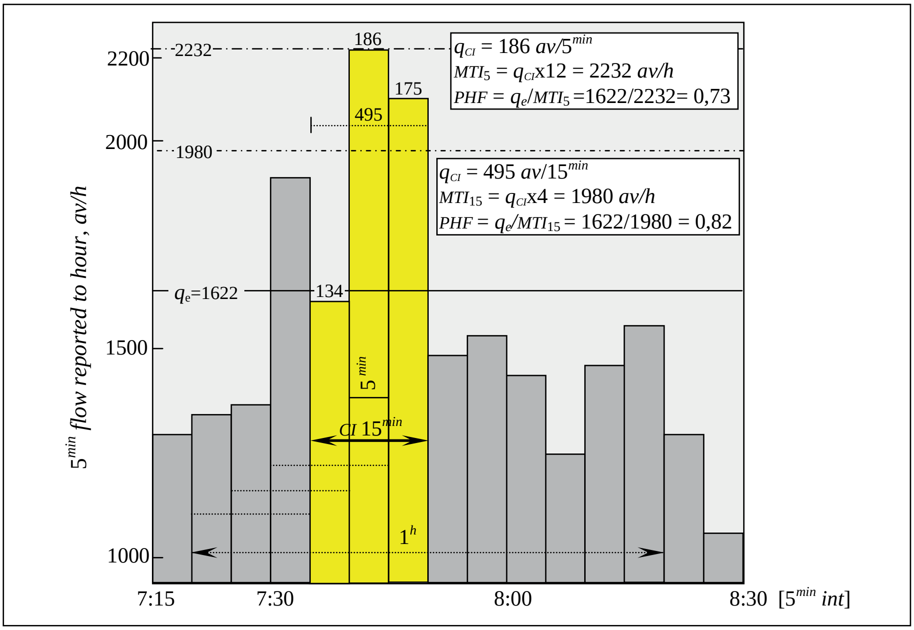

Figure 1. Flow in fractions of an hour of 5^{min} in the max peak hour over 24^h, *AA*.

Designing a road section for the MTI_5 would lead to an oversizing of the road, moreover, assuming the real value of the peak hour as the design flow rate would mean having probable congestion phenomena for several fractions of an hour that would require long times to be disposed of. This suggested choosing the MTI_{15} as a reference value for the design/analysis of roads. In the design of the section, reference can be made to the hourly flow rate even when it is necessary to consider short-term traffic peaks (15^{min}) if they are amplified with the *PHF*.

1.3.3. *The variability of the flow rate*

Since the causes that determine vehicular mobility vary over time (hour and day of the week, season etc.), and in space, the variations in hourly flow will also be: typical is the trend in the hourly volume of traffic on working days for rural and urban roads, which has two characteristic peaks in correspondence with the start/end times of the activities, Fig. 2.

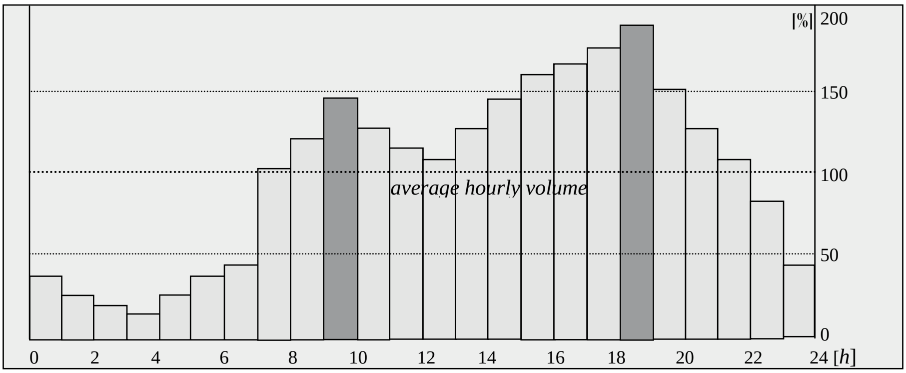

Figure 2. Traffic volumes in a typical day, non urban road, *AA*.

1.3.4. *The n-th peak hour*

If we wanted to describe the phenomenon of the variability of the hourly flows of a road for a long period of time, e.g. a year, we could detect the flow rates of the 8760 hours and report them in a graph where in ordinates we will read the flow rate reached in the corresponding hours indicated in the abscissa. In this characteristic graph, the flow rate reached in the *n*-th hour, which has been exceeded for n–1 hours in a year, is called the flow rate of the *n*-th peak hour (Fig. 3).

Often in the ordinates of the diagram, characteristic for the type and function of the road, the hourly volume is reported in % of the volume of average annual daily traffic (*AADT*) reached or exceeded in the number of corresponding hours in *x*-axis.

The curves of the 200 highest hourly flow rates recorded in a year for different types of roads (Fig. 4), show that the maximum hourly volumes range from 10 (urban) to 45% (touristic road) of the *AADT*; they are also 2 to 12 times greater than the average annual hourly volumes.

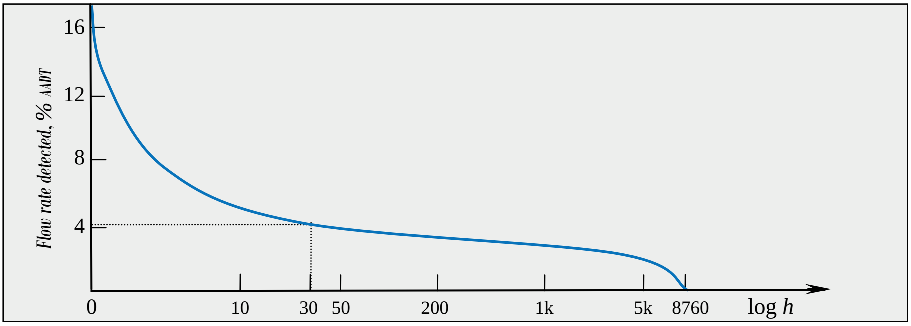

Figure 3. Distribution-type of the peak hours of a whole year, *AA*.

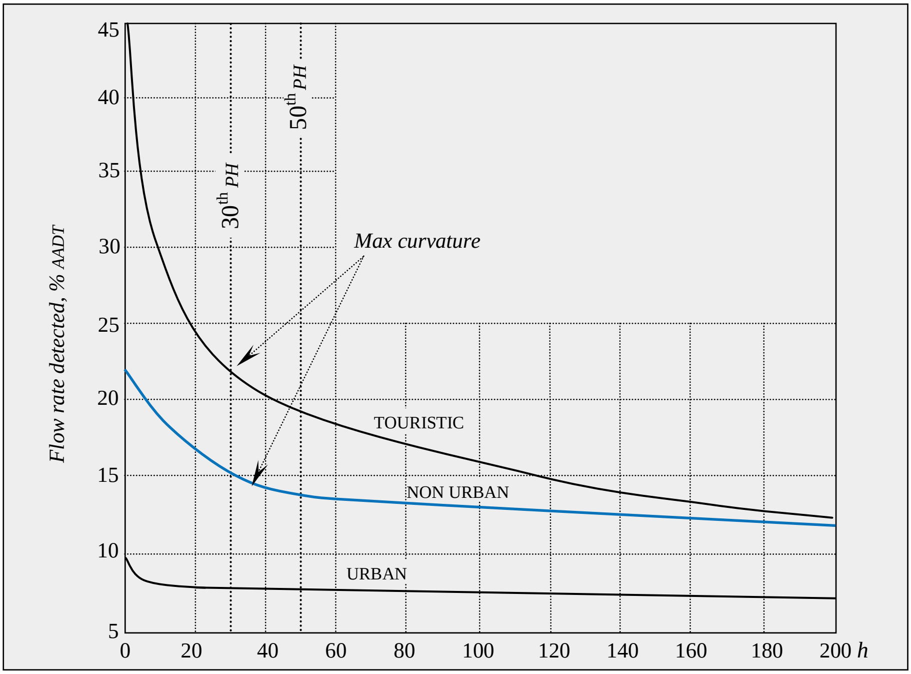

Figure 4. First 200 peak hours by road type, *Matson*.

1.3.5. *The AADT, annual average daily traffic*

The *AADT* (aka *ADT*) is the total volume of traffic over a period of time, in whole days, lasting more than one day and less than a year, divided by the number of days in that period.

In Italy, a census of road traffic is carried out annually on state roads and motorways, which provides for daily counts according to a program of sixteen daytime surveys (7:00–19:00) and seven night surveys (19:00–7:00) distributed on weekdays and seasons according to codified methods.

With the data acquired from the counts, the following forms of *ADT* (Geneva formulas):

$$ADT_{DSS} = ADT \text{ daytime spring/summer}$$
$$ADT_{DAW} = ADT \text{ daytime autumn/winter}$$
$$ADT_{NSS} = ADT \text{ night spring/summer}$$
$$ADT_{NAW} = ADT \text{ night autumn/winter}$$

from which:

$$ADT_D = ½(ADT_{DSS} + ADT_{DAW})$$
$$ADT_N = ½(ADT_{NSS} + ADT_{NAW})$$

at last:

$$ADT = ADT_D + ADT_N \tag{8}$$

Traffic surveys are analytically divided by vehicular categories (from motorcycles to articulated trucks) and by type of traffic (light, heavy); for each category and for each type, the *ADT* day and night; the *ADT* results as the sum of the *ADT* of all vehicle categories and includes traffic in both driving directions [*DD*].

The share of *ADT* corresponding to the hourly flow rate of design is indicated by [*A*] and expressed in decimals; this factor, referring to the $30^{th}/50^{th}$ *PH*, varies with the type and function of the road.

In the design of the road section, both in urban and rural areas, it is sometimes necessary to know the directional distribution of traffic [*D*] which represents the share of traffic in the most heavily loaded direction in the *n*-th *PH* taken as the basis of computing. Indeed, if during a typical working day there are *PHs* in both directions of significantly different magnitude due to commuting phenomena, it will be necessary to take this into account in the section design, determining separately, for each driving direction, the no. of lanes necessary to dispose of the traffic expected at the predetermined level of service (LOS).

In urban areas, on single–carriageway roads, on the other hand, the reversibility of one or more lanes (called trivialized) can be provided to cope with unidirectional flows of demand concentrated in certain time slots; this traffic management measure, used in Europe and North America, is rarely used in Italy.

The factors mentioned A and D are used to determine the directional hourly volume when designing a road section (directional design hourly volume per driving direction, q_{DDHV}) by the expression:

$$q_{DDHV} = ADT{\cdot}A{\cdot}D \quad [ae/h\text{-}DD] \tag{9}$$

ADT = average daily traffic in pax–car equivalent [*ae*] per day [*d*], *ae/d*

A = share of ADT hypothesized as the hourly design volume at the 30th/50th PH, [*ae/h*]/[*ae/d*], decimals

D = share of traffic in the most loaded DD, decimals.

1.3.6. *The design volume*

The choice of the design's hourly volume is not trivial: in fact, choosing that of the annual peak hour would ensure excellent flow conditions but at high costs and the section would certainly be exuberant for almost all the hours of the year; choosing the flow rate of the 200th PH would risk, on the other hand, an undersizing of the section and a number of hours/year (*h/y*) of congestion unacceptable to users.

Since in all the diagrams (Fig. 4) the maximum curvature occurs in a range between the 30th and the 50th PH, the corresponding hourly volume is assumed as the design flow rate, accepting *a priori* 29–49 *h/y* of excess demand and a decay of flow conditions.

It can, however, be said that the choice of the hourly design volume must derive from an analysis of the most accurate possible of the demand combined with an equally precise economic analysis conducted, for instance, by comparing the construction costs with the benefits deriving from the reduction of transport costs for different values of flow rate (which will correspond to different conditions of operation of the road), bearing in mind that the number of hours of probable congestion must be contained in the 30–50 *h/y* where, in practice, the hourly flow rate ranges between 8 and 13% of the ADT expected.

To design the road section, i.e. the number of lanes necessary to dispose of a given demand in a future year ensuring a certain level of service, it is necessary to determine the traffic of year zero (year of entry into operation) and that expected after a certain period of time (useful life), generally twenty years: this traffic, in *ae/h*, will be the reference volume for the design of the road section.

In summary, the basic data necessary for a rough design of a road section are:

— *ADT* to year zero
— *ADT* after twenty years of operation (20^{th} year = year of design)
— diagram of the *n*-th *PH* for the years zero and twenty
— traffic composition for the year zero and twenty
— possible distribution of the volume between *DD* in the 20^{th} year.

1.4. Flow-density relationship

The relationship between flow q and vehicular density k, $q=uk$, is known as the general equation of state of traffic flow; its graphical representation or "fundamental diagram" can be inferred experimentally from traffic surveys by interpolating the data obtained with approximating curves. These curves have a trend of the type shown in Fig. 5.I, of a form similar to that of an inverted parabola.

Since the speed $u_i=q_i/k_i$ is determined by the slope of the vector radius of the point (q_i, k_i), the fundamental diagram illustrates the relationships between all three parameters: q, k and u.

The points of the curve represent a succession of stationarity states; the tangent at a point gives the speed at which the change of state propagates, known as shock wave.

The shape of the diagram is purely qualitative as it is influenced by numerous factors (traffic composition, drivers behaviour, road type and design characteristics, weather conditions etc.) that can change its course.

Considering the flow q as a density-dependent variable, it is logical that it passes through three characteristic points:

1. a null point, for zero density k_0 (no vehicles)
2. another null point, for maximum density k_j (stationary vehicles)
3. an optimal/maximum point (q_o), for a density value k_o, intermediate between 0 and k_j, called capacity.

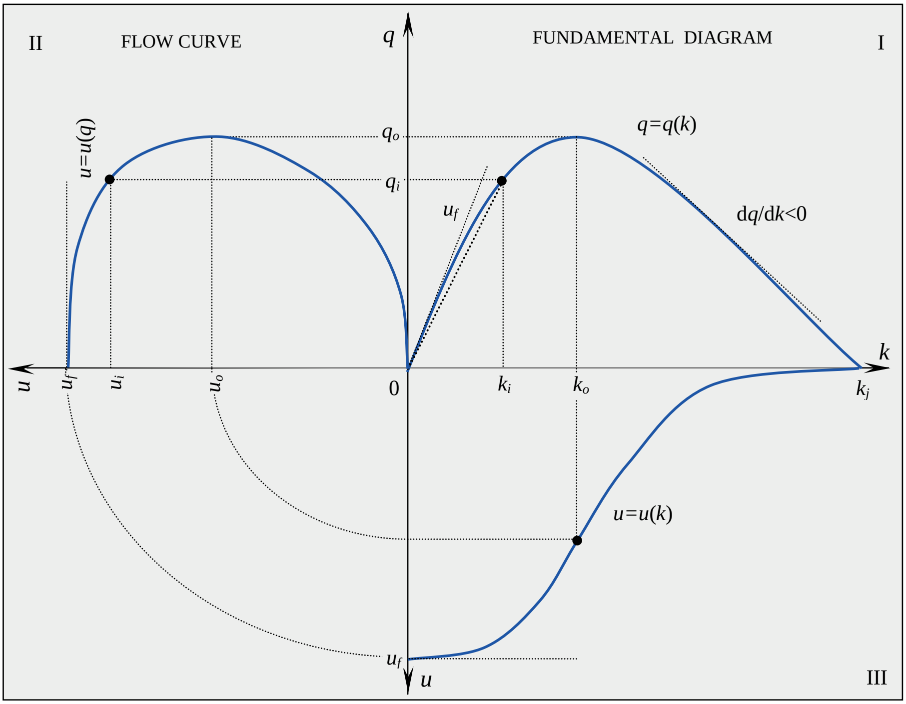

Figure 5. Approximating traffic flow curves: *q–k*, *u–q*, *u–k*, *AA*.

Since vehicular flow, within limits that are not yet rigorously defined, is a substantially random phenomenon, it is not possible to establish the exact maximum of the flow, i.e. the capacity of the road: it could be defined (Haight) as «that value of flow that has a sufficient probability of not being exceeded».

From Fig. 5.I it can be seen that as the density increases from 0 to k_o, the flow rate increases until it reaches its maximum q_o precisely in correspondence with k_o, called optimal density; in this 1st branch the trend of the curve can be more or less concave downwards depending on the rate of increase of vehicles with lower than average speed.

The flow q takes place in stable conditions because the increases in demand can be absorbed by reductions in spacing and speed that allow a new condition of stationarity to be reached, albeit at a lower level than the initial one.

Once the maximum flow conditions have been exceeded, we move on to the conditions of *forced flow* (2nd branch of the curve), characterized by high vehicular densities, rapid decreases in flow due to the combined effect of the reduction of spacings and average speeds: the vehicular stream is progressively compacted, reducing the spacing until it cancels them out for $k=k_j$ (maximum density ≈150 *av/km-ln*) where motion ceases and the formation of queues begins.

1.5. Experimental speed-flow relationship

This relationship can be derived experimentally by averaging the instantaneous speeds of vehicles crossing a road section over a period of time in which the flow is constant, for different values of the latter.

In the plan *u–q* we will obtain a widespread set of points that can be interpolated by a least squares regression curve of the type represented in Fig. 5.II. The curve cuts the ordinate at a point (u_f) which represents the average of the *free flow speeds* (*FFS*), tilts downwards as the flow increases, inasmuch, as demand increases, so do the number of drivers who maintain below–average speeds, affecting the motion of the faster ones.

The increase in flow rate, achieved at the expense of speed decreases, vehicular stream compaction and restrictions on driving freedom, continues until maximum function is reached for an average speed u_o about half of the *FFS* one, approximately 45–55 *km/h*. From that point (capacity), the flow rate can no longer increase and, if the increase in demand continues, the lower branch of the curve (forced flow field) will be traveled, characterized by congestion phenomena (stop-and-go motion) where the flow progressively decreases until it ceases completely. The diagram *u–q* saying flow curve, it is different for road type and, for the same type, it varies with the geometric, traffic and environmental characteristics.

1.5.1. *The u–q statistical diagram*

To better illustrate the *speed–flow* relationship, it is useful to refer to a particular diagram, shown in Fig. 6 where the plane of the figure is divided into three areas delimited by limit curves, the location of the points representing average travel speeds (ratio between the length of a road segment and the time taken to travel it, stops excluded) corresponding to given hourly flow rates, which have a 10 and 90% chance of being exceeded. In practice, 80% of the average speed values measured for a certain flow rate fall within these limit curves.

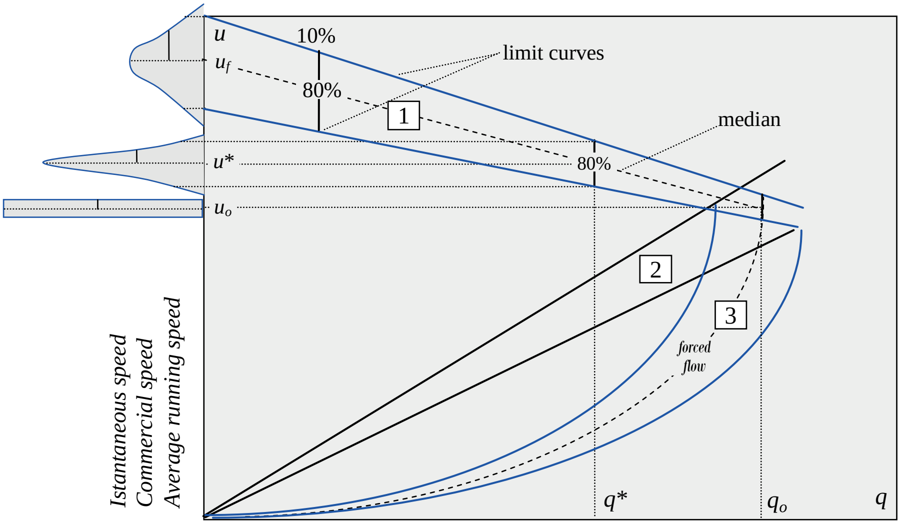

Figure 6. Statistical diagram *running speed–flow rate, AA.*

Zone "1" identifies stable flow conditions: when, for a flow rate q^*, the average speed falls in this zone, the flow occurs at the average speed u^* without exceeding, with 80% probability, the straight lines indicated by the two limit curves.

Zone "2" identifies unstable flow conditions: the corresponding values of q and u they can be kept for short periods of time; both tend to decrease, causing the flow to fall into zone "3" of *forced flow* where congestion phenomena are located. Vehicles disturb each other, low speeds and high densities favor sudden stops of the flow and the generation of queues. It may also happen that, as demand decreases, the queue is quickly disposed of, in which case the driving conditions can be returned to the stable flow zone.

The diagram cited can also be constructed by placing in ordinates the *average travel speed*; it can be defined as the average commercial speed maintained by drivers on a given road segment at a certain hourly flow rate.

For *commercial speed* it means the ratio between the length of a road segment and the time taken to travel it, including slowdowns and/or stops due to traffic lights. However, the diagrams that can be derived with the average commercial speed are analogous to those that can be built by the average running speed.

For design and analysis purposes, we define the "average base speed", weighted average of design speeds of the homogeneous segments