Notes on the Design Concepts for Transport Infrastructures: Past and Future

Michael Bernard

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Abstract

Since their emergence in the 1930s, transport planning and engineering have developed a range of concepts which help planners to design networks and road infrastructures. 30th-hourly volume, peak 15-minute period, average daily traffic etc. Parallely, though not integrated in these approaches, several economically oriented evaluation concepts, such as the cost- benefit analysis, have been conceptualised.

The commonly used techniques for measuring capacity are mainly based on evolutionary concepts. These have been established due to the lack of detailed traffic information and limited capacity to evaluate the data. This situation has changed but is still far from the optimal state of full information, which surely never will be achieved.

The urging questions are: What kind of design concepts are used nowadays and what are they based on? What are the limitations and the assumptions? Are these assumptions still feasible, given the changed traffic situation?

There will not be a simple answer to these questions but the common techniques for measuring capacity need to be observed. In addition to that alternative methods should be shown that could be helpful to gain new design concepts. Other engineering sciences may already use concepts which are also applicable to the traffic system.

Presentation and paper will be structured as followed:

Presentation of common design concepts for transport infrastructure

New design concepts

Outlook

Keywords

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Notes on the Design Concepts for Transport Infrastructure: Past and Future

Michael Bernard
IVT, ETH Zürich
ETH Hönggerberg HIL F12.1
CH-8093 Zürich

Telephone: +41 1 633 66 94
Telefax:
Bernard@ivt.baug.ethz.ch

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1 Introduction

Since their emergence in the 1930s, transport planning and engineering have developed a range of concepts which help planners to design networks and road infrastructures. These early works were focused on the determination of the capacity of freeways, driven by the steadily increasing traffic volume. By now it is accepted, that the construction of traffic infrastructure is needed for the economical development. Originally the authorities' intention was to provide a system that did not restrict the traffic flow. But it was obvious that this philosophy could not match the economic restrictions. To optimise traffic flow, measurements have to be evaluated and combined with mathematical models which describe the coherence.

The commonly used techniques for measuring capacity are mainly based on evolutionary concepts. These have been established due to the lack of detailed traffic information and limited capacity to evaluate the data. This situation has changed but is still far from the optimal state of full information, which surely never will be achieved.

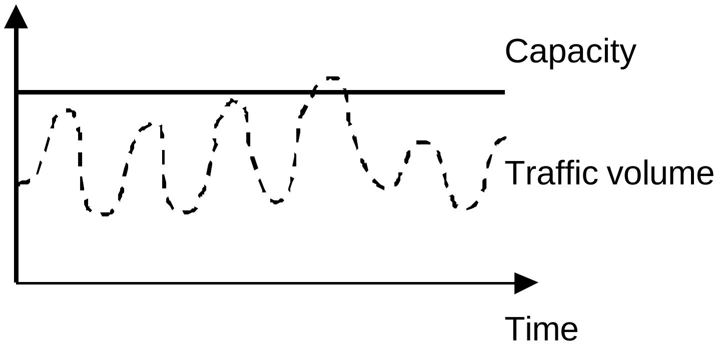
2 Elementary design concepts in transport science

Publications until 1950 mainly deal with the description of traffic flow. Following distances and traffic volumes dependent on mean speeds were identified as fundamental coherence. Driven by the increasing availability of automatic traffic counters, the results became more and more detailed.

By creating fundamental diagrams it was possible to estimate the capacity for given road types. The design traffic load was identified by pattern over time. This design concept was adopted from other engineering sciences (Figure 1). Generally speaking a resistance of a system (capacity) should be greater or equal to the load (traffic volume).

As a result of the steadily increasing traffic volume it was no longer practicable to build roads for the maximum expected traffic volume, for the costs of constructing roads for a rare event are too high. Consequently the design criterion was modified and the traffic load was allowed to be higher than the roads capacity for a given number of hours per year.

Figure 1 Elementary Model: Resistance (Capacity) and Load



The capacity of roads are determined by measurements of traffic volumes. In contrast to other engineering sciences the system does not break as a result of the load itself. There are also some random effects which influence the breakdown probability. In addition to that even the capacity could not be compared with some kind of constant resistance. E. g. light and weather circumstances are significant effects.

3 Design concepts till 1950

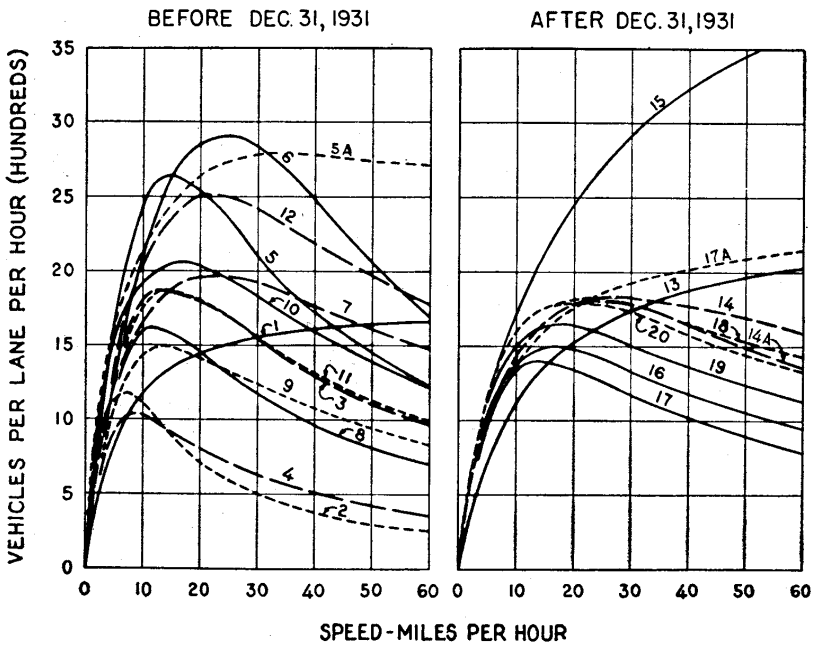
In Bureau of Public Roads (1950) the design of highway capacity is compared with the "effectiveness of the various facilities in serving traffic". In addition to that the shape and the run of roads, the drivers attributes and structural measures have to be considered. The variables mean speed, relative interference by the vehicles and the traffic volume were regarded as quality criteria. Most of the publications till 1950 define the capacity of a lane by the actual mean speed and the following distance (Table 1 and Figure 2). It becomes obvious that there are big differences within the results of the listed capacities, calculated by various researchers.

Table 1 Early sources of calculated capacities

No.	Author	Year	Safety distance in feet, speed in miles/hour			Re-action-time	speed. in miles at q_{max}	Max capacity in veh./h
1	Schwanter	1924		2.933v+	14.7	2.00	∞	1800
2	French periodical	1924	$0.337v^2$+		14.7	0	6.6	1190
3	Lewis	1925	$0.0742v^2$+	0.733v+	14.7	0.50	14.1	1870
4	Schaar	1925	$0.213v^2$+	1.47v+	14.7	1.00	8.3	1050
5	Johnson	1926	$0.0667v^2$+		15	0	15.0	2640
5a	"	1928	$0.5v^{2.3}$+		15	0	34.3	2800
6	Kelker	1926			17.6	-	25.0	2900
7	Highway Research	1927	$0.0366v^2$+	1.10v+	17	0.75	21.5	1970
8	Weninger	1929	$0.109v^2$+	0.733v+	14.7	0.50	11.6	1610
9	Ehlgotz	1929	$0.0773v^2$+	1.47v+	14.7	1.00	14.2	1490
10	Daugherty	1930	$0.0556v^2$+	0.733v+	15	0.50	16.4	2060
11	New York Regional	1931	$0.072v^2$+	0.733v+	15	0.50	14.4	1870
12	Allan	1931	$0.0333v^2$+	0.733v+	14	0.50	20.5	2520
13	Johannesson	1933		2.20v+	25	1.50	∞	2400
14	Massachusetts WPA	1934	$0.025v^2$+	1.47v+	20	1.00	28.3	1830
14a	"	1934	$0.0102v^{2.3}$+	1.47v+	20	1.00	24.1	1800
15	Greenshields	1935		1.10v+	21	0.75	∞	4800
16	Birula	1935	$0.0635v^2$+	1.47v+	16.4	1.00	15.8	1500
17	Nevins	1939	$0.0833v^2$+	1.47v+	16	1.00	13.9	1400
17a	"	1939		2.20v+	16	1.50	∞	2400
18	Clayton	1941	$0.0333v^2$+	1.47v+	15	1.00	21.2	1830
19	"	1941	$0.05v^2$+	1.47v+	15	1.00	17.3	1650
20	Gulstad	1941	$0.0373v^2$+	1.47v+	15	1.00	19.8	1790

According to Bureau of Public Roads (1950)

Figure 2 Calculated Capacities of car-following models



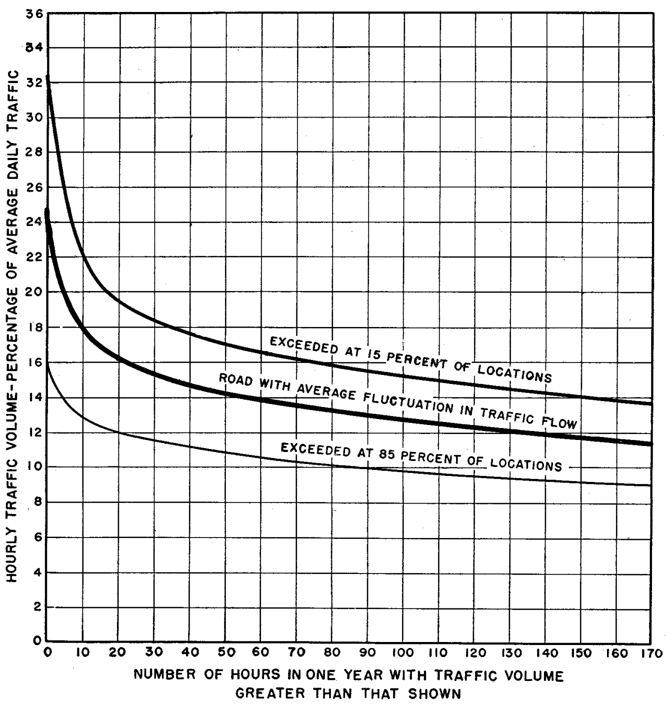
Source: Bureau of Public Roads (1950)

Data from automated measurements were available for the analyses of the Highway Capacity Manual (1950) (Bureau of Public Roads, 1950). It is remarked that the capacities of roads varies over few years while neglecting structural measures. This development was explained by the improvements of cars and drivers experiences. But it was assumed that there will be no further development of these effects. During observations 28% of the drivers chose following distances at which they would not have had the chance to avoid an accident in case of the preceding vehicle breaking hard.

The design concept of multi-lane roads in Bureau of Public Roads (1950) was described by the mean speed of the vehicles. Having a high traffic volume, the speeds tend to homogenise in a way that the differences in speed of the vehicles nearly vanish. This state was identified as the critical density at which the maximum traffic volume was realised under prevailing circumstances. A further increase in density causes a reduction in speed and traffic flow. The maximum capacity is assumed to be valid for all roads of the same type under ideal conditions and free traffic flow. For the design process the maximum capacity is reduced by factors which describe type and alignment of the road and other structural facilities. In Bureau of Public Roads (1950) is mentioned that slopes do not effect the capacity.

As shown above it is not feasible to construct a road for the maximum predicted traffic volume, i. e. capacity overloads are accepted. The 30th highest hour is named as a practical criterion in Bureau of Public Roads (1950). The given reason for this concept is a steep slope in the graph of the sorted hours of traffic volume towards hours with stronger traffic volumes. This should be the point of a good cost-benefit ratio (Figure 3).

Figure 3 Relation between peak hour flows and annual average daily traffic



Source: Bureau of Public Roads (1950)

4 Diversification and introduction of levels of service

Hitherto concepts based on intensive measurements to identify the maximum occurred volume. Qualitative characteristics in combination with parameters had to be found to describe the traffic flow. The basic attributes are car-following distances, mean speed and – by combining both – the traffic flow.

These concepts do not show explicitly a separation of load and resistance of the system. The resistance of the system could be compared with the maximum capacity that was identified by the maximum traffic flow observed for one hour. The expression “maximum capacity” is misleading in this context as it is actually the maximum observed traffic flow. Design concepts in general describe a maximum resistance (capacity) as the mean of a series of strain test of the system till its breakdown. This is a major problem in transport engineering, as it is not possible to run tests under ideal conditions with a controlled traffic load. Nevertheless factors regarding structural facilities and alignment effects have been identified. These variables reduce the maximum capacity under ideal conditions. The load is adapted by not concerning for example 30 hours with the year's highest volume. This is equivalent to some kind of reciprocal safety coefficient, for it lowers the safety instead of increasing it. But the raw traffic flow describes the load of the system not in every way. Therefore it is seen that a breakdown occurs not only because of a high traffic flow but also while the flow is relatively low.

While Bureau of Public Roads (1950) mainly uses means to describe the traffic flow, later studies tried to explain variability by adding new variables and categories. Improvements could be found in Highway Research Board (1965). The Poisson distribution is used to describe the distributions of time spacing between vehicles as a function of the traffic volume. A further extension of the existing models is the introduction of levels of service (LoS), regarding the observed coherence that traffic flow is dependent on the actual volume and density. In Highway Research Board (1965) six levels of service are defined as follows (see also Figure 4):

A) "Level of service A describes a condition of free flow, with low volumes and high speeds. Traffic density is low, with speeds controlled by driver desires, speed limits, and physical roadway conditions. There is little or no restriction on maneuverability due to the presence of other vehicles, and drivers can maintain their desired speeds with little or no delay."

B) "Level of service B is in the zone of stable flow, with operating speeds beginning to be restricted somewhat by traffic conditions. Drivers still have reasonable freedom to select their speed and lane of operation. Reductions in speed are not unreasonable, with a low probability of traffic flow being restricted. The lower limit (lowest speed, highest volume) of this level of service has been associated with service volumes use in the design of rural highways."

C) "Level of Service C is still in the zone of stable flow, but speeds and maneuverability are more closely controlled by the higher volumes. Most of the drivers are restricted in their freedom to select their own speed, change lanes, or pass. A relatively satisfactory operating speed is still obtained, with service volumes perhaps suitable for urban design practice."

D) "Level of service D approaches unstable flow, with tolerable operating speeds being maintained though considerably affected by changes in operating conditions. Fluctuations in volume and temporary restrictions to flow may cause substantial drops in operating speeds. Drivers have little freedom to maneuver, and comfort and convenience are low, but conditions can be tolerated for short periods of time."

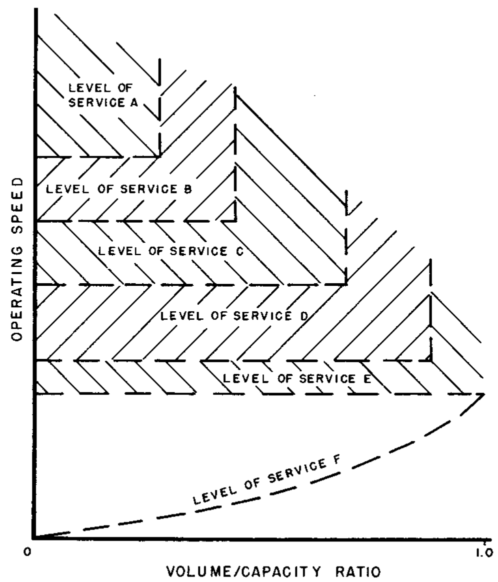
E) "Level of service E cannot be described by speed alone, but represents operations at even lower operating speeds than in level D, with volumes at or near the capacity of the highway. At capacity, speeds are typically, but not always, in the neighbourhood of 30 mph. Flow is unstable, and there may be stoppages of momentary duration."

F) "Level of service F describes forced flow operation at low speeds, where volumes are below capacity. These conditions usually result from queues of vehicles backing up from a restriction downstream. The section under study will be serving as a storage area during parts or all of the peak hour. Speeds are reduce substantially and stoppages may occur for short or long periods of time because of the downstream congestion. In the extreme, both speed and volume can drop to zero."

If the distributions of velocity published in Bureau of Public Roads (1950) and Highway Research Board (1965) are compared, the changes during this period become obvious. The differences of the years 1950 and 1965 in Figure 5 could only partly be reasoned by modified measurement techniques. Improvements of the infrastructure and vehicles in addition to larger traffic volumes have affected curve progression.

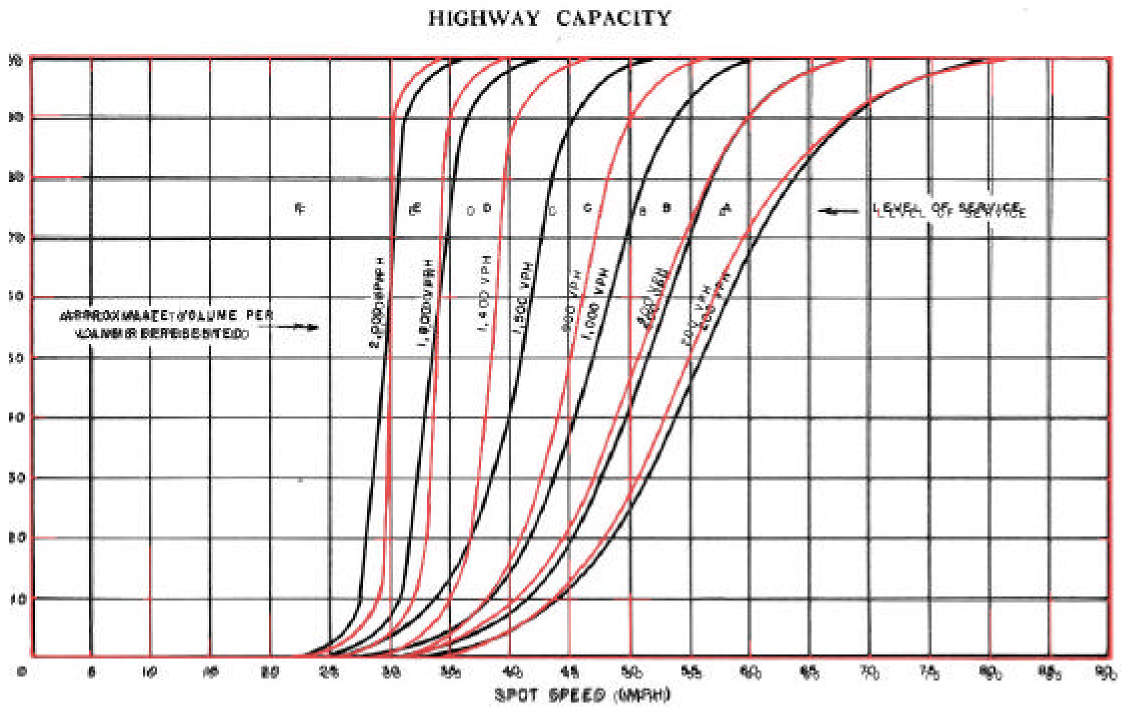
The observed behaviour varies from country to country and in time periods of few years. The concentrated efforts of continuous counts made it possible to publish modified distributions of headways and loads to regularly provide fundamental diagrams for many countries.

Figure 4 Levels of Service (LoS) in Highway Research Board (1965)



Source: Highway Research Board (1965)

Figure 5 Comparison of distributions of speed in the USA 1950 (black) and 1965 (grey)



Source: Bureau of Public Roads (1950), Highway Research Board (1965)

5 Design concepts

Important Variables that influence the traffic volume are divides into two basic categories by Dietrich (1964): traffic or environmental influence and individual driving behaviour. It is said that the individual behaviour could not be determined, as the drivers reactions could not be predicted or measured. In this context the driving experience, the drivers actual constitution and the purpose of the trip should be named. Dietrich (1964) assumes that there are various interacting effects that influence the chosen speed and following distance in that way that is might be impossible to assign the observed behaviour to each variable. But the traffic and environmental effects could be categorised and often even be measured. These effects are listed in Table 2.

Table 2 Impacts of external effects on traffic volumes

Impact	Characteristics
Vehicle	Type (Car, heavy-goods vehicle, motorbike, bike)
	Dimensions
	Acceleratio n ability
	Climbing ability
Modal split	Proportion of heavy-goods vehicles
Road	Alignment
	Cross-section profile
	Grades
	View distances
	Lateral constructions
Weather	Rain
	Snow
	Fog
Lighting conditions	Back light (glare)
	Twilight
Traffic regulation	Speed restrictioni
	Signal control
	Priority rule

According to Dietrich (1964)

5.1 30th hourly volume

Design concepts commonly use the 30th hourly volume, described in Bureau of Public Roads (1950). The 30th hourly volume was chosen because of a flattening of the slope at this point. Antusch (1981) adds that this property could only be observed for censuses that mainly consist of commuter and weekday traffic. On weekends an incline of the slope could be found at the 30th hourly volume. But nevertheless even for these types of traffic the 30th hourly volume should be used as a design measure, because hours with higher traffic volume are affected by incidents like accidents and special weather conditions. After the 20th or 30th hourly volume these events could be excluded (Anusch, 1981). In addition to that economic issues support this concept, as a design for hours of larger volumes will lead to a low utilisation ratio. The 30th hourly volume could be described as the volume which is greater or equal to 99.66% of the hourly volumes of a year. But this expression is rarely used in practice.

The actual publication of the Highway Capacity Manual (2000) (Transportation Research Board, 2000) suggests the design for the 30th to 100th hourly volume. The practical traffic volume for each direction could be calculated using the proportion of the direction of the hourly peak volume and the average annual daily traffic. These are multiplied by K-factors ranging from 0.091 to 0.1 which describe the grade of urbanisation.

5.2 Alternative concepts

Dietrich (1964) defines fife criteria that could be measured to identify the practical traffic volume or the capacity of a road under ideal conditions. In this publication alternative characteristics for design concepts are presented.

- Time headway

  If a gap distribution follows the Poisson distribution, the cumulative gaps become even within a graph with logarithmic scale of the cumulative percentages. It is assumed that a critical gap that identifies congested traffic flow is identified by an offset of this line. As this method bases on only one parameter, it could not be used to define a capacity.

- Velocity

  In Bureau of public (1950) it is assumed that the practical capacity it achieved under ideal conditions if 72% of the vehicles could not reach their desired velocity. Based on this Dietrich (1964) presumes that the proportion of interferences increases linearly with the traffic volume. I. e. there is no indicator to find a point which characterizes the capacity.

- Increase in travel time

  Analyses of Dietrich (1964) have shown that travel times increases continuously with the traffic volume. Defining a factor that represents the maximum acceptable increase in travel time would be a random value.

- Overhaul

  It is assumed that the number of desired overhauls increases with larger traffic volumes whereas the possibilities are decreasing. In Bureau of Public Roads (1950) it is said that the capacity of a road is reached when 50% of the desired overhauls could not be carried out. In this context it seems problematic to define the number of desired overhauls and possibilities to calculate the ratio.

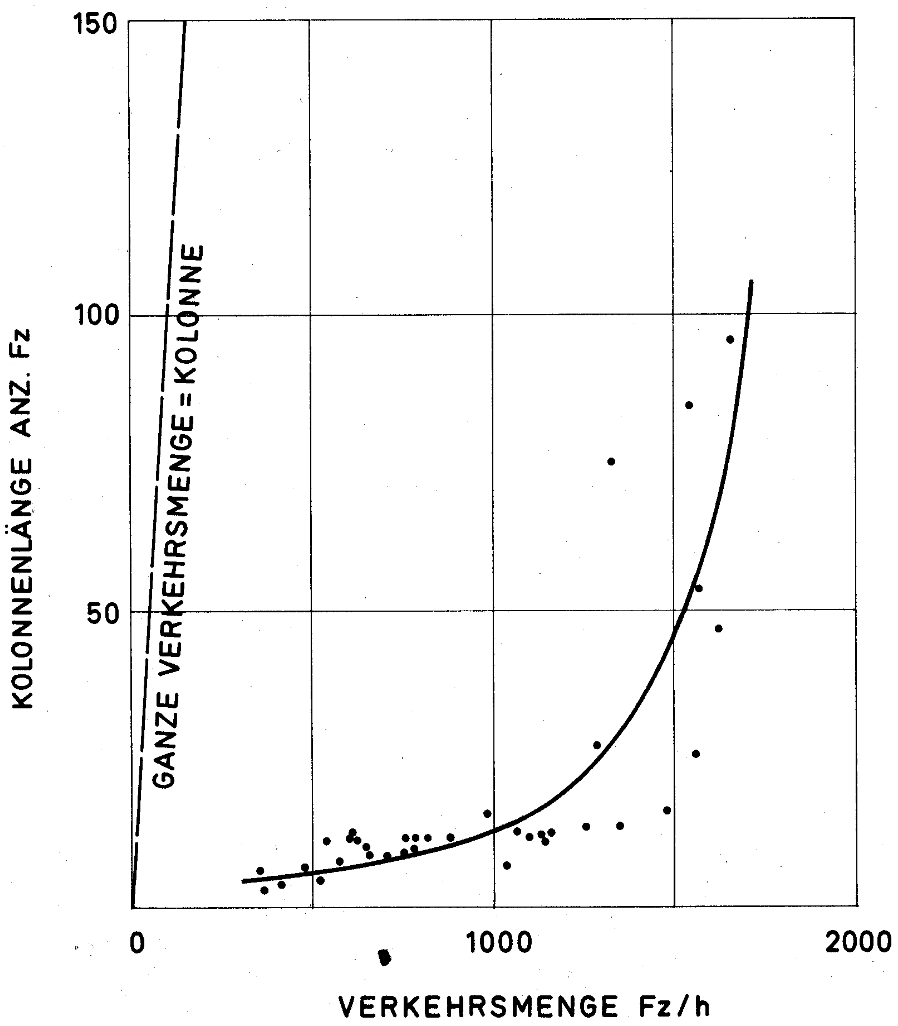
- Platooning

  Dietrich (1964) proposes to use platooning as a measure for interferences. The impact of the interference could be described by the size of the platoon, the gaps and its velocity. Each of the platoons are weighted and linked with the corresponding traffic volumes. The weight of a platoon (Pk) is defined as follows:

$$P_k = \frac{K^2}{V_m} \frac{9 - \Delta t}{\Delta t}.$$

In this formula K represents the number of vehicles of the platoon, Vm the mean velocity of the platoon and Δt the gap. In addition to that it is assumed that vehicles with a gap greater than 9 seconds and driving faster than 100 km/h are not interfered by other vehicles. The results are show in Figure 6.

Figure 6 Largest platoon vs. traffic volume



Source: Dietrich (1964)

5.3 Models using the fundamental diagram

The fundamental diagram is commonly used to define the capacity to a road and the levels of service. Measurements are interpolated using mathematical models. Classically a continuous model is assumed that describes all combinations of traffic volume and speed. Recent studies propose a non-continuous model of different states. Exemplarily the differences in the continuous model by van Aerde and the model of Transportation Research Board (2000) should be shown.

5.3.1 Continuous model of van Aerde

Van Aerde (1995) describes a car-following model by a gap (h) between two vehicles dependent on three constants (c_1, c_2, c_3), the free flow velocity (v_0) and the actual velocity (v). It is assumed that the traffic density (k) becomes denser at small gaps. The gaps (h) are defined as follows:

$$h = c_1 + \frac{c_2}{v_0 - v} + c_3 v$$

and the results for the density (k) are:

$$k(v) = \frac{1}{c_1 + \frac{c_2}{v_0 - v} + c_3 v} \ .$$

May and Keller (1968) and Zackor et al. (1988) have shown that the curved shape of this model (Figure 7) aligns well with given measurements. But in practice this formulation leads to long term when solved for velocity (v) or traffic flow (q), as remarked by Ponzlet (1996):

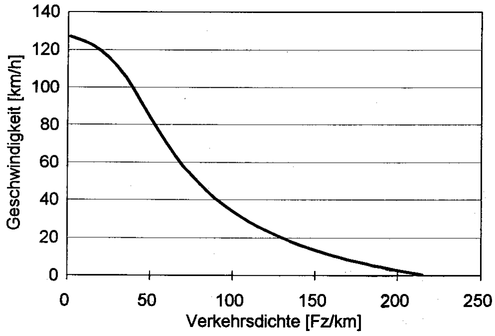
$$v(q) = \frac{1}{2}\left(v_0 + \frac{c_1 q}{1 - c_3 q} \pm \sqrt{R} \right)$$

$$\text{with } R = \frac{v_0^2 - 2c_1 q v_0 + 2c_1 c_3 q^2 v_0 + c_1^2 q^2 - 4c_2 q + 4c_2 c_3 q^2 - 2c_3 q v_0^2 + c_3^2 q^2 v_0^2}{(1 - c_3 q)^2} \ .$$

The maximum traffic flow q_{max} could be calculated, having R=0 as follows:

$$q_{\max} = \frac{-2\sqrt{c_2}\sqrt{c_2 + c_1 v_0} + c_1 v_0 + 2c_2 + c_3 v_0^2}{c_1^2 + 4c_2 c_3 + 2c_1 c_3 v_0 + c_3^2 v_0^2}$$

Figure 7 Continuous speed-density model by van Aerde



Source: Ponzlet (1996)

If the fundamental diagram should be presented using a continuous model, the on of van Aerde could be used, as it could be well adapted to the measurements by defining the parameters c_1, c_2, and c_3. The measuring period has to be regarded. Long intervals (e. g. hourly volumes) could neglect peaks, whereas short intervals may lead to over-estimations of the maximum traffic flow. Keller and Sachse (1992) suggest five-minute intervals as trade-off. Mean states could be represented and peak values will be observed. Brilon and Ponzlet (1996) have developed a formula to convert different intervals:

$$\frac{q_{\mathrm{int}erval}}{q_{60}} = a - bq_{60} ,$$

with q_{interval} traffic flow of measured interval,
q_{60} traffic flow of a 60-minute interval,
a, b factors of Table 3.

Table 3 Factors a and b to convert 60-minute intervals

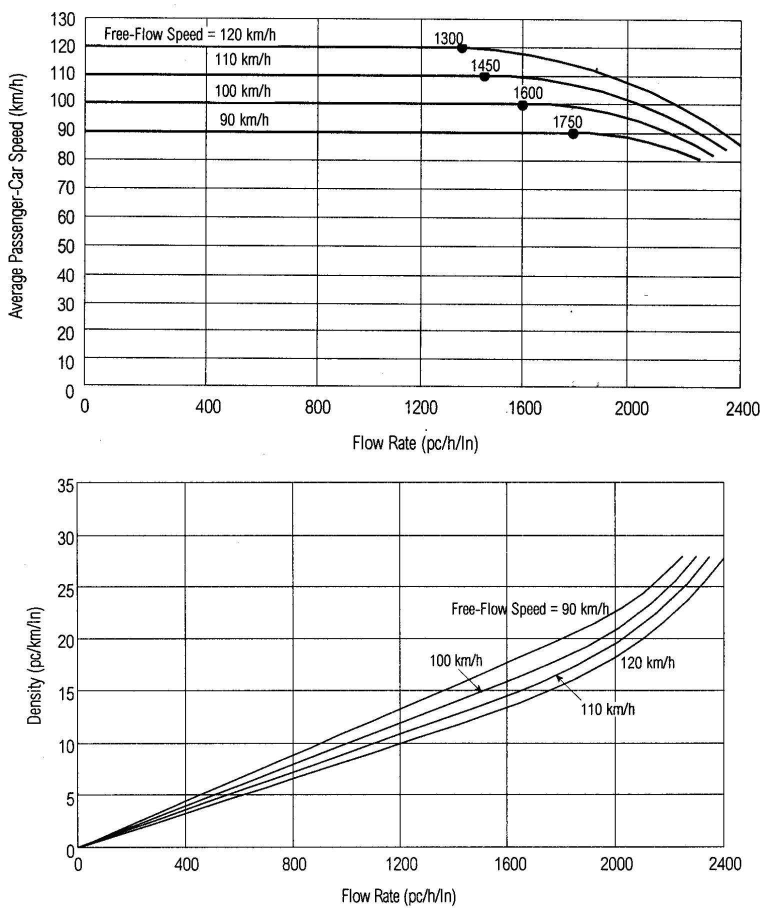
Interval	*a*	*b*	r²	Standard-deviation
1 min	2.469	$3.45 \cdot 10^{-4}$	0.49	0.17
5 min	1.335	$4.79 \cdot 10^{-5}$	0.06	0.09
15 min	1.090	$1.41 \cdot 10^{-6}$	0.00	0.05

Source: Brilon and Ponzlet (1996)

5.3.2 Multipart models

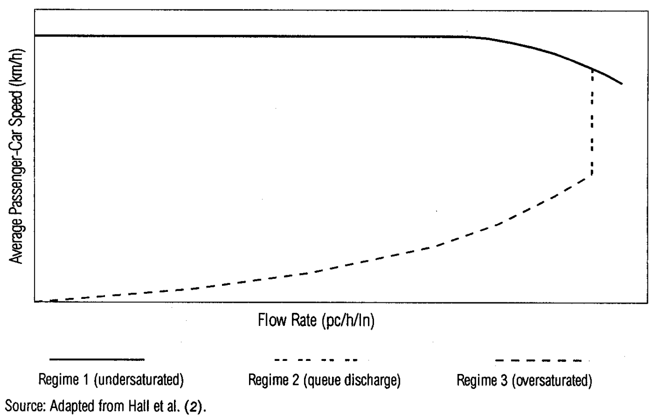
Recent analyses do not support a continuous model of the fundamental diagram but split shape of two states. Measurements done by Brilon and Lemke (1999) assist this type of model which is also shown in the actual Highway Capacity Manual (2000) (Transportation Research Board, 2000; see Figure 8). In Figure 9 three states of traffic flow are defined. The undersaturated flow is not affected by any up- or downstream incident. The traffic volumes are relatively low and speeds range from 90 to 120 km/h. Larger traffic volumes will result in speeds between 70 and 100 km/h. The state "queue discharge flow" could be seen behind a bottleneck with a stable traffic flow of 2000 to 2300 vehicles/hour per lane at speeds higher than 55 km/h. A bottleneck downstream leads to oversaturated flow. The changeover form free flow to congested flow in this model is not continuous. In Transportation Research Board (2000) this state change is combined with a capacity drop.

Figure 8 Fundamental diagram in Transportation Research Board (2000)



Source: Transportation Research Board (2000)

Figure 9 Queue discharge and oversaturation

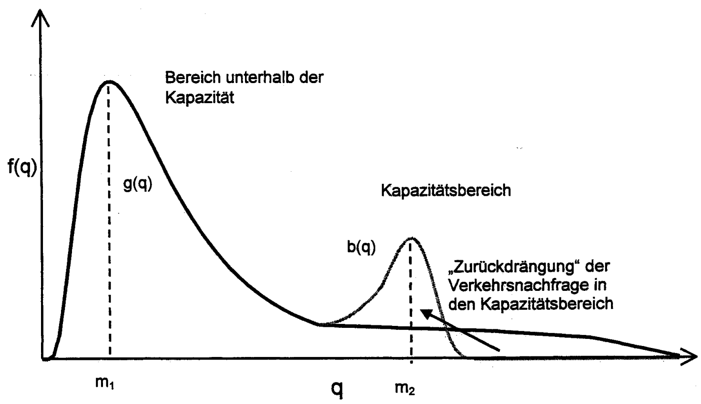


Source: Transportation Research Board (2000), according to Hall et al. (1992)

5.4 State of the art: design capacity

If time series analysis are done, two maxima in the frequency distribution over traffic flow could be remarked (Cohen, 1983). The first accumulation is assumed at low traffic volumes and is driven by demand. The second accumulation could be seen at large traffic volumes and characterizes the maximum volumes of the observed section. The mean of the second accumulation (seen as a distribution) is assumed to describe the maximum capacity (Figure 10).

Figure 10 Distribution of traffic volumes by Minderhoud et al. (1996)



Source: Brilon (2003), according to Minderhound et al. (1996)

Analyses of Brilon (2003) do not support this theory, as the mean of the second accumulation of the frequency distribution describes values which are much lower than the assumed capacity of the roads. Hempsey and Teply (1999) propose a method to define the design capacity which is similar to the concept of the 30^{th} hourly volume. The 30^{th} hourly volume meets with criticism, as there is no information about the states of traffic flow and the quality of service when large traffic volumes or congestion is assumed. A statement being of the form that 90% of the vehicles could use a road at a quality of service at least of level B makes more sense. This type of approach allows to do cost-benefit analyses, as information about levels of service is given.

Matt and Elefteriadou (2001) have identified the breakdown probability for the 401 highway in Toronto. They define that a breakdown is observed if the mean speed of all lanes drops for fife minutes below a critical speed that separates free flow from congested traffic. In this publication it becomes clear that the capacity cannot be seen as a fixed value but as a random variable. With higher traffic volume the probability of a breakdown becomes higher. But there is no maximum traffic volume that cause a breakdown when the volume is increased by one unit.

In Brilon (2003) a method is suggested to identify the capacity of roads, that focuses on the traffic volumes of measuring intervals that are followed by a capacity drop. These intervals are defined by a reduction of the mean speed of the following interval. Based on the statistical evaluation of lifecycles the product-limit method is used to estimate the random variable of capacity. Intervals with free flow traffic and no breakdown in the following interval are marked as “survived” intervals. This method takes into account that a capacity drop is not only dependent on the current traffic volume but in addition to that of random effects. Here and as well for other method must be assured that an observed breakdown is not a result of a high density downstream. Therefore bottlenecks should be considered for measuring when these methods are applied.

6 Conclusion

In practice new techniques in defining the design capacities and different levels of service are rare used. But it becomes more and more clear that capacity could not be seen as a fixed value under given conditions but as a random variable. Analyses support this assumption, as breakdowns are not only provoked by large traffic volumes. A distribution of the random effects or at least a critical value that describes a probability of breakdown is to be found. This should be connected to the levels of service that regards the user’s and operator’s costs. Other important issues are traffic safety, environmental sustainability and costs. In addition to that there is an impact of the demand side. The users will do their travel decisions bases on the assumed traffic volumes and congestion probabilities, i. e. that there will be an influence concerning departure time and route choice.

The goal should be to integrate these effects into *one* design concept. The distributions of the coefficients have to be regarded to do risk analyses, which includes interactions of incidents. A design concept based on mean values could not provide the desired reliability having heterogeneous transport.

7 Literature

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