Highway Sizing

JOSEPH D. CRABTREE AND JOHN A. DEACON

A critical examination is made of the conventional method for highway sizing, that is, the determination of lane requirements. Ranked hourly traffic-volume distributions, obtained from 1977 Kentucky volume stations, are examined to test certain assumptions common to the conventional approach. Assumptions regarding the existence and location of "knees" within these distributions, a common requirement of current procedures, are found to be of questionable validity. However, the fundamental fallacy of the conventional procedure rests with its focus on a single design hour and its orientation toward conditions experienced by the highway rather than by the user. This can readily be overcome by basing size decisions on an alternative criterion such as the percentage of vehicles that suffer congestion during the design life. An example demonstrating this concept is presented. At the same time, more significant improvement in sizing methodology can be achieved by directly computing the economic efficiency of investment in additional lanes. An example is presented to demonstrate current capabilities for such computations. The example also demonstrates that current procedures do not always yield the most economical designs and that the most economical highway size is affected by the specific form of the traffic-volume distribution. Use of economic efficiency analysis as a standard tool in evaluating important sizing decisions is highly recommended.

In many highway construction or reconstruction projects, one important decision is the number of lanes to be provided. Procedures used to determine lane requirements (highway sizing) are normally based on identification of a single design hour within which the anticipated demand volume [commonly the 30th highest hourly volume (HHV) in the design year] is balanced against supply volumes (capacities or service volumes) for the alternative highway sizes under consideration.

During the past three decades, conventional highway-sizing procedures have remained virtually unchanged. During this same period, other highway decisionmaking processes have changed markedly as emphasis has highlighted broad social concerns and environmental impacts and as competition for the public dollar has intensified. In view of this situation, it is appropriate to reexamine conventional sizing procedures. The project reported here was initiated to determine the soundness of these procedures and to identify, if necessary, possible techniques for improvement.

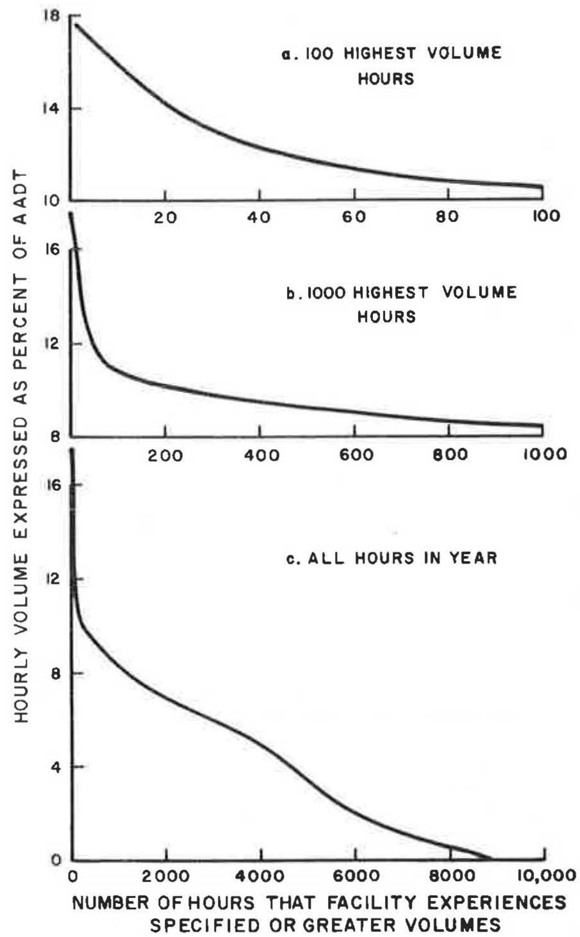
CURRENT METHODOLOGY

Development of the current sizing methodology is credited to Peabody and Normann. In 1941, by using the single design-hour volume versus capacity, they recommended use of a design-hour volume within the range of the 30th to the 50th HHV (1). Endorsements for use of the 30th HHV soon came from the American Association of State Highway Officials (AASHO) and the Committee on Highway Capacity of the Highway Research Board. AASHO, in 1945, adopted the 30th HHV for a year 20 years from the date of construction as the design-hour volume for the national system of Interstate highways, an adoption that, with only slight modifications, has remained in subsequent design standards (2). In 1950, the Committee on Highway Capacity recommended use of the 30th HHV as the normal design-hour volume (3). However, the Committee cautioned, as had Peabody and Normann, that the 30th HHV was not necessarily applicable in every instance and that it would "not always result in the best engineering practice" (3).

To understand the rationale for these recommendations, it is necessary to visualize the characteristic shape of the plot of a ranked hourly volume distribution. Figure 1a, constructed from hourly volume data obtained from one automatic traffic recorder (ATR) in Kentucky during 1977, is one such plot. The resulting curve seems to show a "knee" (a small region with a rapid change in slope) at or about the 30th HHV. After observing the regularity with which such a knee occurred in the region between the 30th and 50th HHV for a large number of highway locations, Peabody and Normann concluded that it was impractical to design for volumes greater than the 30th HHV and further that designs for volumes less than the 50th HHV would likely result in only small savings in construction cost but great loss to the expedition of traffic movement (1). Through the years, use of the 30th HHV seems to have been based to a large degree on the assertion that it yielded the most economical design, or, as stated by the Committee on Highway Capacity, it is at this point that the "ratio of benefit to expenditure is near the maximum" (3). Matson, Smith, and Hurd more subjectively argued (4): "The most equitable ratio between the service provided by the road and its costs will be achieved when the design volume is selected near the knee of the curve."

Although endorsement of the 30th-HHV design concept by these respected authorities contributed

Figure 1. Typical ranked hourly volume distribution (station 16).



greatly to its rapid and widespread adoption, at least one other factor was also of importance. The Committee on Highway Capacity had concluded that the 30th HHV, when expressed as a percentage of the annual average daily traffic (AADT) volume, changed very little from year to year (3). A future-year AADT prediction could be easily and accurately converted to the design-hour volume through application of what has come to be called the K-factor (the frequently measurable ratio of the 30th HHV) to the AADT. Confidence of the designer in the design-hour volume prediction was thus greatly enhanced.

The most authoritative current recommendations for highway sizing are those of the American Association of State Highway and Transportation Officials (AASHTO). AASHTO recommends use of an hourly volume representative of flows at the end of the design life, that is, 10-20 years following completion of construction. For rural highways with normal flow variations, the 30th HHV should be used. For rural highways with unusual or highly seasonal traffic fluctuation, the design hourly volume should be as follows (5):

> about 50 percent of the volumes expected to occur during a very few maximum hours of the design year....A check should be made to insure the expected maximum hourly traffic does not exceed possible capacity.

For urban streets and highways, the design hourly volume should be the average of the 52 highest afternoon peak-hour volumes for each of the weeks in the design year. After observing that this average is not significantly different from the 30th HHV, AASHTO concluded (6):

> Therefore, for use in urban design the 30th highest hourly volume can be accepted since it is a reasonable representation of daily peak hours during the year. Exception may be necessary in those areas or locations where concentrated recreational or other travel during some seasons of the year results in a distribution of traffic volume of such nature that a sufficient number of the hourly volumes are so much greater than the 30 HV that they cannot be tolerated and a higher value must be considered in design.

CRITIQUE

To evaluate the soundness of sizing procedures, one would prefer to examine a large number of past sizing decisions and determine, in retrospect, the fraction that was successful. Unfortunately, such an evaluation is very difficult, if not impossible, both because of the difficulty of acquiring the necessary data and because of the absence of an accepted criterion for defining success. The approach taken in this critique is therefore to focus on the identification of procedural difficulties and on an assessment of the validity of assumptions that undergird the decisionmaking process.

In applying the conventional procedure, the designer is continually challenged to determine when the 30th or 50th HHV should be used (for normal flows) or when other more appropriate measures should be sought (for unusual flows). This choice is one of increasing difficulty: There is simply a continuum of traffic-flow patterns reflecting the wide variety of travel desires served by individual facilities and their varying degrees of operational adequacy. Not only does this difficulty raise questions about procedural technique, but also an analysis of flow patterns suggests possible fallacies in underlying assumptions.

The conventional highway-sizing procedure draws its strength in part from the following four basic assumptions: (a) the ranked hourly volume distribution exhibits a discernible knee; (b) this knee occurs at or near the 30th HHV; (c) the knee defines the point of most economical sizing; and (d) the 30th HHV, expressed as a percentage of the AADT, remains constant over time.

To examine the first two of these assumptions, traffic-volume data collected in 1977 from 45 Kentucky ATR stations were analyzed. Three ranked hourly volume-distribution graphs for each station, similar to those of Figure 1, were constructed for use in the visual component of the analysis. Although most prior analyses had examined in detail only the 200 or so highest volume hours, the three different data sets were used here to identify any possible bias in the more conventional but also more limited examination.

The first portion of the analysis was a subjective one. Four observers were asked to independently examine each ranked hourly volume-distribution graph and to determine whether a knee could be discerned. They were told only that a knee was a small region on either side of which the slopes of the curve were markedly different. Figure 1 is typical of the situation in which there was general agreement among the observers that knees did exist. In Figure 1, the four observers located knees on the 100-h, 1000-h, and 8760-h graphs within the following ranges in ranks, respectively: 23rd-25th HHV, 70th-84th HHV, and 100th-200th HHV. Figure 2 is representative of graphs for which the observers had more difficulty locating knees. Three of the four observers were unable to locate knees on the 100-h and 1000-h graphs, and two did not find a knee on the 8760-h graph. The difficulty with the graphs in Figure 2 was that the curves, although well behaved, exhibited slopes that changed quite gradually with increases in rank. Any knee was therefore very difficult to identify.

The first part of Table 1, which summarizes this portion of the analysis, shows that there was a discernible knee in most instances and that the likelihood of finding a knee increased as the size of the data set increased. However, in a substantial percentage of cases (approximately 16 percent for the 100-h graphs), no knee could be found: These cases cannot be dismissed as mere exceptions. Also noted, although not shown by Table 1, is the fact that there were many cases in which individual observers disagreed over the existence of knees. Assuming that the observers were reasonably competent, this type of disagreement effectively demonstrates the subjective and somewhat vague nature of the knee-of-curve concept.

Observers were also asked to determine, where possible, the location of each knee. This subjective analysis was augmented by a more objective one employing a nonlinear regression program of the Statistical Analysis System (SAS). SAS was used to fit a segmented model to each set of volume data. This involved the optimal separation of each set of data into two subsets and the fitting of independent curves to each of the two subsets. Figure 3 typifies the results. The knee was assumed to occur at the intersection of the two fitted curves, the location labeled "boundary" in Figure 3. The remarkable similarity between the observer-reported knee locations and those determined by SAS gave much credibility to the SAS analysis. Although both linear and quadratic models were tested, they were found to yield similar boundary locations, and only results from the quadratic models are reported here.

The results of the analysis of knee-of-curve location are also summarized in Table 1. The first striking observation is that the location of the knee is influenced drastically by the extent of the data set. This fact became readily apparent early in the research when graphs for individual stations were compared (see, for example, Figure 1): It was confirmed by both the visual and SAS analyses when the average ranks of Table 1 were determined. The sensitivity of the location of the knee to the amount of data is sufficient to cast serious doubt on the efficacy of knee-of-curve procedures. A knee the location of which varies for a given data set with the method for graphically portraying that data would seem to be of questionable reliability.

Figure 2. Ranked hourly volume distribution showing indistinct knee (station 46).

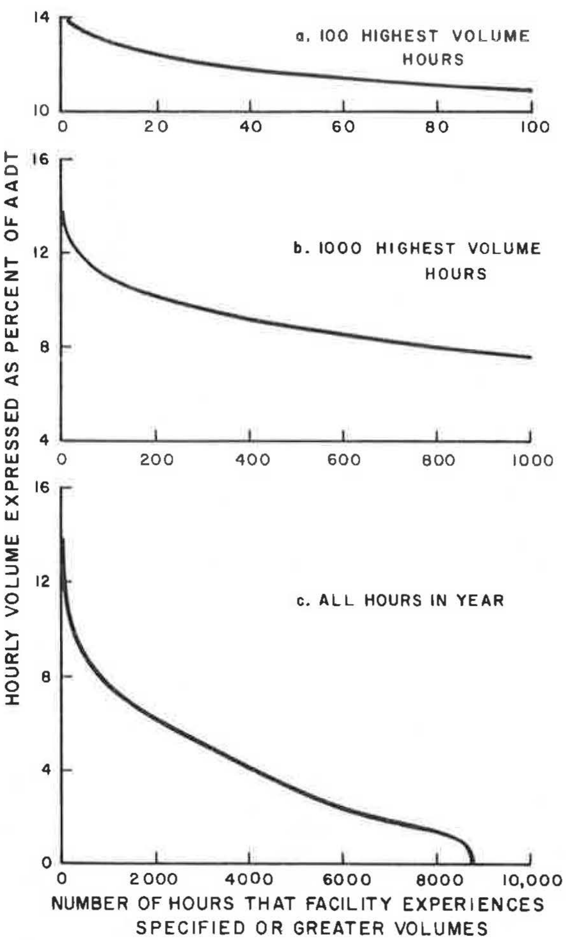


Table 1. Existence and location of knee for ranked hourly volume distributions.

Item	Graph of 100 Highest Volume Hours	Graph of 1000 Highest Volume Hours	Graph of All Hours in Year
Percentage of graphs with discernible knee (total for four observers)	83.8	86.2	91.2
Average rank of hour at knee location			
Range for four observers	6.6-9.9	47-82	310-620
Segmented model	19	110	360
Percentage of knee locations within 30th-50th HHV interval			
Average for four observers	0.6	33.3	0.0
Segmented model	11.1	0.0	0.0

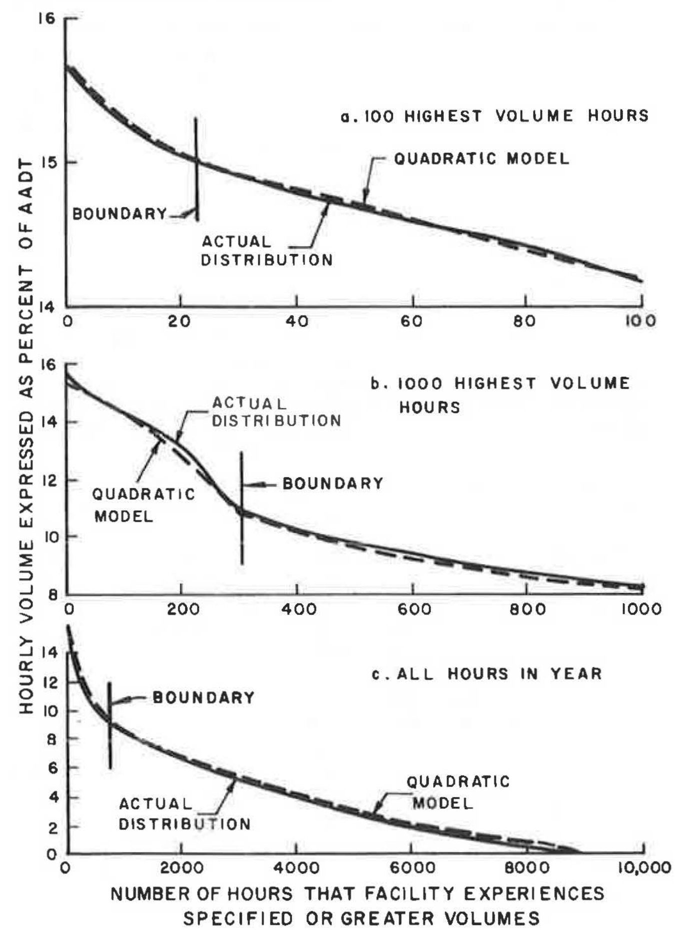
Originally there was great interest in whether the knee occurred at or near the 30th HHV: Interest waned when it was conclusively established that the knee location was influenced by the number of hours within the data subset. A quick glance at the average ranks in Table 1 suggests that by selection of some subset of data between the 100 and 1000 highest volume hours, the location of the knee would average at or near the 30th HHV. At the same time, Table 1 shows that most of the knees were located outside the accepted range of the 30th-50th HHV for the data groupings employed here.

There was also much variability from station to station in the location of the knee. Results of the visual observations of the 1000-h graphs are shown in the following tabulation:

Location of Knee	Percentage of Stations
None	14
Between	
1st and 20th HHV	16
21st and 40th HHV	20
41st and 60th HHV	15
61st and 120th HHV	20
121st and 300th HHV	10
300th HHV and above	5

Certainly, those using knee-of-curve sizing proce-

Figure 3. Typical ranked hourly volume distribution showing segmented quadratic model of best fit (station 7-SB).



dures would be well advised to determine the location of the knee of curve for each situation rather that to assume that it lies within the 30th-50th HHV range. This recommendation supports earlier work of Werner and Willis (7), who showed that the knee was not necessarily located at the 30th HHV and that it tended to lie within the 200th-600th HHV range for the larger AADTs.

A third assumption implicit in the conventional sizing procedure is that the knee defines the point of most economical sizing. Unfortunately, it has been impossible to conclusively prove or disprove this assumption. There is certainly an intuitive appeal to the argument that as one considers volumes to the left of the knee, construction costs would increase greatly while only a very few more hours or users would be accommodated. As one considers volumes to the right of the knee, very little is likely to be saved in construction cost but much would be sacrificed by the user as many additional hours would become congested. At the same time, it seems obvious that such a conclusion might be seriously distorted by focusing, as has been common in the past, on the few heaviest volume hours (perhaps 200) in some year 10-25 years in the future. In effect, the design to accommodate the future-year 30th HHV is very similar to the design to accommodate the maximum hourly volume in the design life, a design that most designers would consider to be inappropriate and uneconomical. Further to the point of economy in highway sizing, no study has been discovered in which any tests have been made or other objective evidence presented that supports the assumption that the knee defines the point of most economical sizing. At the same time, it is possible to demonstrate, as is done later in this paper, specific examples for which the knee does not define the most economical size.

The fourth assumption that has been important to widespread adoption of the conventional sizing procedure is that the 30th HHV, expressed as a percentage of the AADT, remains constant over time. Following such an assertion by the Committee on Highway Capacity in 1950 (3), a number of important studies have shown that the K-factor is not invariant and typically decreases with the increasing volumes that often accompany the passing of time. Among these studies are those of Walker (8), Bellis and Jones (9), Reilly and Radics (10), Chu (11), and Cameron (12). With this rather conclusive analysis, it was not imperative to examine the matter fully during this investigation. A superficial examination was made, however, of data from Kentucky ATR stations for 1973 and 1977. Between 1973 and 1977, the K-factor decreased for 28 of the 40 common ATR stations, increased for 8, and remained the same for 4. The average K-factor for the 40 stations decreased during this period from 11.5 to 11.2 percent. It is obvious, therefore, that the K-factor for a specific highway location is a time-variant quantity.

Conventional sizing procedures have been used with much success for many years, they are viewed quite favorably by design agencies, and their widespread use is likely to continue for many years. Those continuing to use these procedures, however, should consider implementation of changes suggested by the above analysis. The design-hour volume should be selected at the knee of the ranked hourly volume-distribution graph rather than at some arbitrarily chosen point such as the 30th HHV. In addition, the graph should contain all hourly volume data collected throughout the year rather than some arbitrarily chosen subset such as the 200 highest volume hours. Finally, since the pattern of traffic flow is likely to be different from location to location, each site must be individually analyzed to ascertain what volume corresponds to the knee and how the K-factor is likely to vary through time. Other improvements, as identified and addressed in the following section, should also be considered for adoption.

EXTENSIONS

In examining the highway-sizing literature, two promising extensions to the conventional procedure were discovered. Because of their relative ease of implementation and because they overcome certain valid objections to the conventional procedure, they are described in this paper and their use is illustrated by means of examples. Hourly traffic-volume distributions used in these and subsequent examples are shown in Figure 4 and other traffic characteristics are described as part of the list of assumptions for the economic analysis given in the next section. The standard traffic distribution of Figure 4 is representative of the 1977 median for Kentucky ATR stations, whereas the alternate represents 1977 data for one particular station chosen because the hourly flows were less variable than those for the standard. Both distributions have K-factors of 11.2 percent, the 1977 median for Kentucky ATR stations.

The first extension, attributed to Glauz and St. John (13) and reported by the Institute of Traffic Engineers (ITE) Technical Council Committee 6F-2 (14), suggests a user orientation to design instead of the traditional facility orientation. The focus here becomes the percentage of time that the typical user experiences high-volume conditions rather than the percentage of time that the facility experiences such conditions. In the traditional approach, the highway is sized so that it will be congested no more than 30 h during the year or about 0.34 percent of the time. In the user-oriented approach, the highway would be sized so that the user would experience congestion no more than some acceptable percentage of the time. The difference between these approaches derives from the fact that a proportionally greater number of users travel during high-volume hours as compared with low-volume hours.

Figure 5 show the first 200 h of the traffic volume data of Figure 4 replotted to convert from number of hours to percentage of time and extended to show the difference between the user and facility orientations. To modify the conventional sizing procedure to the user approach requires use of ranked volume distributions for users rather than for facilities. The ITE report (14) describes the procedure in some detail. An individual plot, similar to that of Figure 5a, could be used to select a knee to support a specific design decision or a large number of such plots could be examined to locate the characteristic position of a knee or to otherwise derive an acceptable decision criterion.

The user approach is conceptually superior to the traditional one in that it more nearly recognizes the primary purpose of many highway developments--to provide an improved level of service to the road user. Practically, as suggested by Glauz and St. John (13), it offers a superior way to recognize and emphasize peculiar characteristics of recreational and other routes that have peaked-flow characteristics.

A second useful extension to the conventional sizing procedure derives from work of DeVries (15), also reported by ITE (14). To demonstrate the significance of DeVries' contribution, it is first necessary to emphasize that the conventional procedure is based on the concept of a single design hour. Lane requirements are determined by comparing the

Figure 4. Ranked hourly volume distributions for examples.

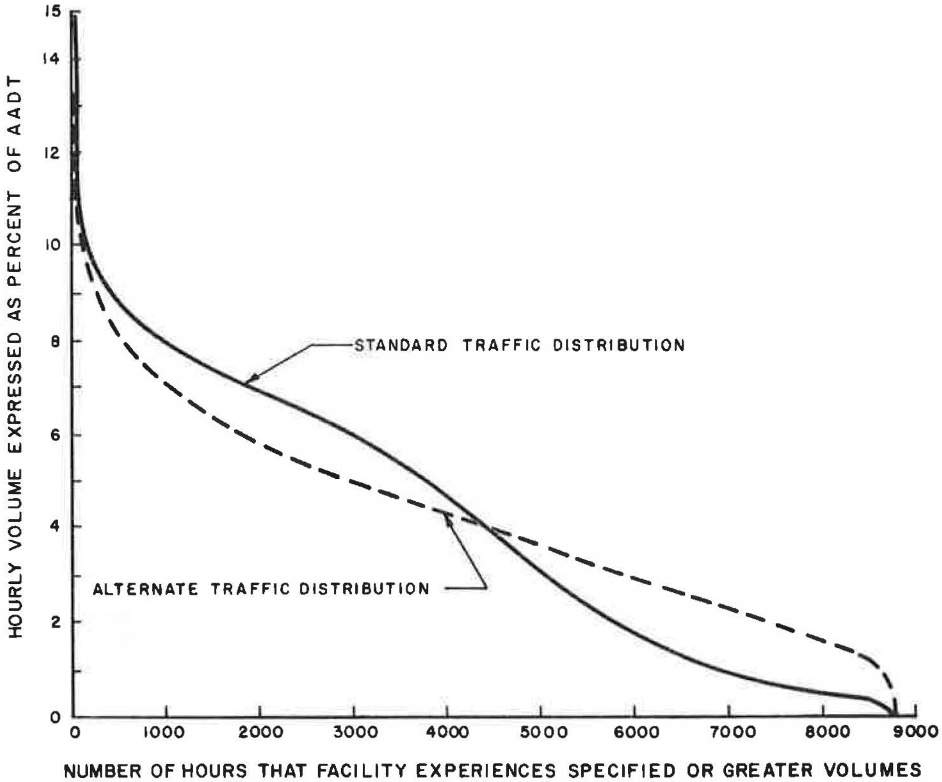
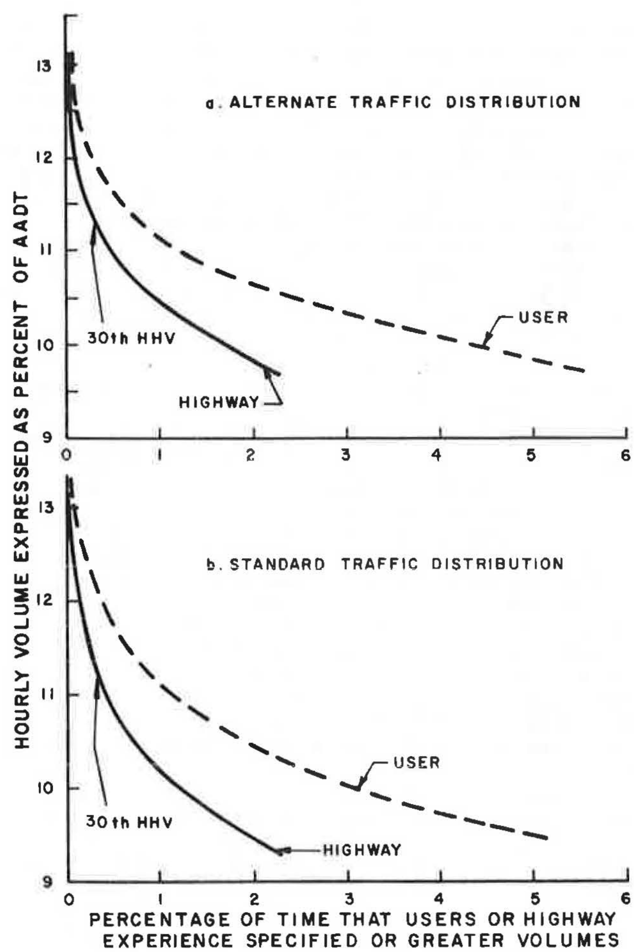


Figure 5. Ranked hourly volume distributions for both users and highway.



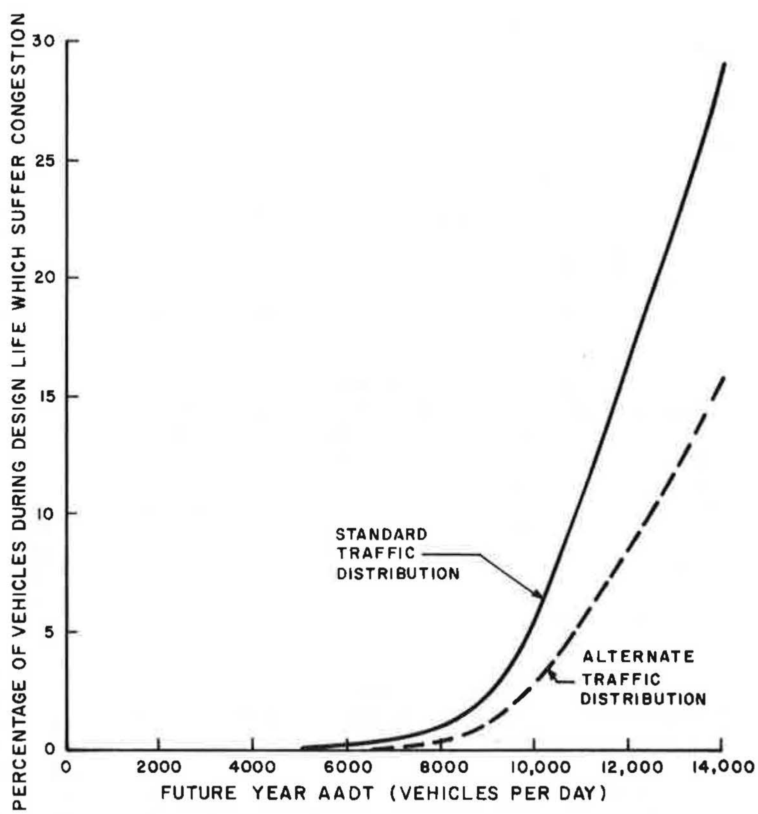
demand volume (design-hour volume) with the supply volume (service volume or capacity) for one particular hour during the entire design life of the highway. Is it not presumptuous to ignore conditions occurring during that overwhelming portion of the design life in which flow is more or less congested than in the design hour? Is it not also presumptuous to base such a design on demand and supply volumes that have been rather arbitrarily selected on the basis of the designer's intuition as to what conditions are acceptable to the traveler and what conditions result in the most economical design? Questions such as these lend credence to attempts to expand the focus from a single hour to a range of hours within the design life.

DeVries suggested that more prudent investment decisions for independent project analysis might result from investigations of the range of top hours (perhaps the highest volume 500) that were encompassed within the desired level of service. This concept might be implemented in any of several ways, which include specification of a minimum number of the top 500 h that must be included within the desired level of service.

As a variation of the DeVries proposal, which includes the Glauz-St. John user emphasis, sizing decisions might be based on the percentage of vehicles during the design life that suffer congestion. A simple but reasonable way to define congestion is in terms of operating conditions representative of D-, E-, or F-levels of service. The objective would be to base size decisions on a congestion level acceptable to the design agency. Figure 6 illustrates the output of such an analysis.

Figure 6 shows the traffic volume that would be subject to congestion on two-lane roadways for a range of future-year AADTs and the two different traffic distributions described earlier. Similar analysis showed that no congestion would be anticipated on four-lane facilities with volumes no greater than a future-year AADT of 14 000. The spe-

Figure 6. Influence of traffic volume on congestion of two-lane sample highway.



cific criterion for highway sizing in this example would have to be selected by the designer. Alternatives might be no congestion, some fixed level of congestion such as 2 percent, or even the knee of the curve. The knee is reasonably well defined in this example, and if that should prove to be true in other circumstances as well, the knee might furnish an acceptable heuristic decision point.

In summary, design to accommodate a single hour in the design life of a facility masks the reality of variable operational-flow conditions through time. This difficulty can and should be overcome by broadening the analysis to include a much larger time frame. Use as the decision criterion of the percentage of vehicles during the design life that suffer congestion accomplishes this objective as well as that of properly focusing on the user rather than the facility. Further testing and use of such a criterion seems warranted.

RECOMMENDED PROCEDURE

Highway-sizing decisions rank among the more important decisions confronting the designer or planner. Differential construction costs are measured in the hundreds of thousands of dollars per kilometer, and the cost of an additional pair of lanes will, in some circumstances, almost double construction outlays. Because of their importance, sizing decisions merit very critical analysis and should not be based on hunch and intuition. Although the conventional procedure can certainly be improved as indicated above, to accomplish what is really necessary requires a completely different perspective on the sizing task.

We contend that sizing decisions should be reached in the same manner as other major investment decisions. In whatever way has been found to be acceptable to each responsible agency, the gamut of both favorable and unfavorable consequences of the sizing decision needs to be identified and evaluated. One such consequence that is often evaluated in public decisions involving the allocation of scarce resources is the economic efficiency of the investment. Economic analysis seems tailor-made to the sizing decision, since the primary impacts are often limited to savings to the road user and costs to the highway agency.

The technical literature abounds in information regarding economic analysis and its application to highway investment decisions. Maring (16) and Hutchinson (17) were among those who specifically advocated use of economic analysis in highway-sizing decisions. Although both presented useful examples to demonstrate their recommendations, effectiveness of these examples was limited by the data that were readily available when their work was performed. Publication of the authoritative manual on user benefit analysis by AASHTO (18) has helped to eliminate many of the earlier constraints to effective analysis. At the same time, it must be emphasized that economic analysis still involves a number of very important assumptions, any one of which can possibly affect the decision. Sensitivity analysis is a recommended technique for assessing the potential significance of the critical assumptions.

To demonstrate application of economic analysis, a hypothetical situation was defined in which a sizing decision was required on a new, 16.1-km highway. Future-year AADT was varied and two ranked hourly volume distributions, as shown in Figure 4, were independently investigated. Details of the analysis are identified below:

Traffic:

1. Growth of 3 percent compounded annually;
2. Composition of 85 percent cars, 10 percent single-unit trucks, and 5 percent combination trucks;

3. Directional split of 55 percent in direction of greatest flow; and
4. Ranked hourly volume distributions as shown in Figure 4.

Roadway:

1. Uninterrupted flow in rural area;
2. Design speed of 96.6 km/h and speed limit of 88.5 km/h;
3. Length of 16.1 km with 3.66-m lanes and 3.05-m shoulders;
4. No access control but four-lane highway has median;
5. Paved surface;
6. Rolling terrain with 11.3 km level, 3.2 km on a 1 percent grade, and 1.6 km on a 2 percent grade;
7. Tangent sections for 11.3 km and horizontal curvature of 1 and 2 degrees on lengths of 3.2 and 1.6 km, respectively; and
8. 100 percent of two-lane highway with passing sight distance in excess of 460 m.

Analysis:

1. 25-year period of analysis;
2. All costs expressed in constant (1975) dollars;
3. Discount rate of 5 percent;
4. Hourly time costs of $3.00 for cars, $7.00 for single-unit trucks, and $8.00 for combination trucks;
5. Construction costs of $615 000/km and $957 000/km for two-lane and four-lane highways, respectively;
6. Maintenance costs of $2660/(km•year) and $4320/(km•year) for two-lane and four-lane highways, respectively;
7. Residual value of $349 000/km and $560 000/km for two-lane and four-lane highways, respectively; and
8. Accident costs of $10.03/1000 vehicle-km and $8.78/1000 vehicle-km for two-lane and four-lane highways, respectively.

Insofar as practical, recommendations and data given by AASHTO (18) were used. Construction and maintenance costs were estimated on the basis of the Kentucky experience, and accident costs as reported by AASHTO (18) were used.

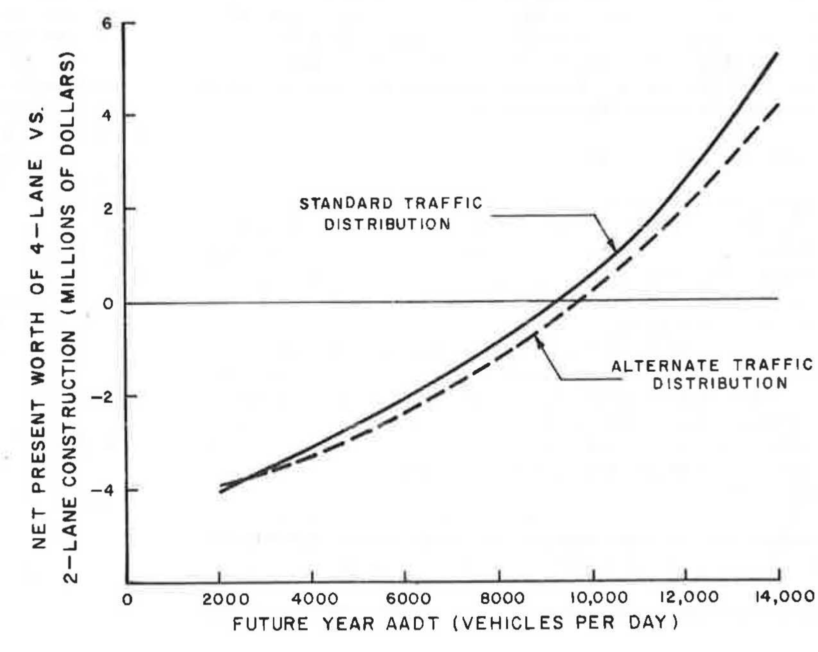
The criterion chosen to represent economic efficiency was the net present worth (NPW) of four-lane as compared with NPW of two-lane construction. Benefits of the four-lane construction included savings in travel time and accident costs and an increase in the residual value of the investment. Greater costs for the four-lane facility were attributed to those of construction and maintenance as well as to increased vehicle operating costs occasioned primarily by increased speed.

Figure 7 summarizes the analysis. For the standard traffic distribution, two-lane construction is seen to be preferable for future-year AADTs less than about 9300 vehicles per day. This break-even volume increased to 9800 vehicles per day for the alternative traffic distribution. The fact that two different traffic distributions, although they have identical K-values and design hourly volumes, had different break-even volumes suggests that factors other than the location of the knee of the ranked hourly volume-distribution curve also influence the most economical design.

A comparison was also made between the break-even volumes of Figure 7 and those determined by the conventional sizing procedure. In the latter case, the break-even volume depends on which level of service is selected to represent acceptable congestion in the design hour. The future-year, break-even AADTs for the conventional analysis were determined to be approximately 4500, 7400, and 9300 vehicles per day for B, C, and D service volumes, respectively. Results from the conventional analysis and the economic analysis thus become comparable only for a level of service (D) that has normally been thought to be intolerable for all but exceptional design purposes. The conclusion, therefore, is that, for this sample problem and a rather wide range in future-year AADTs, the conventional sizing analysis would lead to a design decision different from that of an economic analysis. Of course, specific numbers reported here are unique to the given conditions, and generalizations based thereon are to be avoided.

The example of this section has demonstrated application of the techniques of engineering enonomy to the highway-sizing decision. It has also identi-

Figure 7. Economic efficiency of four-lane versus two-lane construction in sample highway.



fied at least one situation in which the conventional sizing procedure yields a decision different from one based on the criterion of economic efficiency. We are convinced that techniques and data for performing competent economic analyses are readily available and are becoming more and more sophisticated. Further, we are convinced that the economic efficiency of additional-lane investments is one impact that should never be neglected in the sizing decision. At the same time, we are aware that other impacts are sometimes of paramount importance. Who cannot describe a situation in which a nearby cemetery, a row of stately shade trees, a bordering park, or any of a number of other situations has served to constrain the size of a highway improvement? The point is simply that economic efficiency, albeit important, is only one of the many impacts of the sizing decision that must be evaluated if prudent decisions are to be reached.

SUMMARY AND CONCLUSIONS

A critical examination has been made of the conventional method for highway sizing, that is, determination of lane requirements. Although this method has served admirably in the past, improvements can readily be made that will lead not only to more informed but also to more easily defensible decision-making.

The fallacy of the conventional method, which determines lane requirements by balancing a design-hour volume (demand) against service volumes for the alternative highway sizes (supply), rests with its focus on a single design hour as well as with its orientation to the facility rather than the user. It does not explicitly consider, therefore, the normal reason for increasing highway size, namely, the benefits that accrue through time to the users.

Further, some of the basic premises on which the conventional sizing methodology is based have been found to be invalid. Many ranked hourly volume distributions (nth-highest-hour plots) do not exhibit discernible knees, or small regions within which their slopes change markedly. Of those that seem to exhibit knees, knee locations vary among observers and are unquestionably and most inappropriately influenced by the number of hours of volume data being examined. Further, knees usually lie outside the normally accepted 30th-50th HHV interval. Traffic-volume data reported in this paper offer support to the prior conclusion of others that, at a given location, the K-factor (ratio of 30th HHV to the AADT) cannot be expected to remain constant through time and for underutilized facilities typically decreases as traffic volume increases. Finally, the conventional sizing methodology, although it has minimal data requirements and is quite simple to apply, cannot be expected to necessarily yield the most economical highway size decision.

Similar care and attention should be given to decisions regarding highway size as to other major highway investment decisions. The entire gamut of differential impacts, including such factors as the degradation of parks and historic places, aesthetics, and noise and air pollution, should, if possible, be evaluated. Of particular importance to this evaluation is the economic efficiency of the highway investment.

The capacity for using conventional highway economic analysis to aid highway-sizing decisions is well developed and readily available for immediate implementation. Its use is strongly recommended as a rational and defensible basis for supporting sizing decisions. However, for those who find this recommendation unacceptable, other improvements to the conventional methodology are suggested. The first involves focusing on the user instead of the facility by appropriately changing the abscissas of the ranked hourly volume-distribution plots and selecting the design-hour volume at the position of the relocated knee. The second would be more significant but would require a conceptual transition from a single-hour to a range-of-hours approach. A suitable decision criterion in this situation appears to be the percentage of vehicles during the entire design life that suffer congestion for the alternative highway sizes. A decision to increase highway size would be justifiable when the percentage of vehicles suffering congestion on the smaller facility was considered unacceptably large by the design agency.

ACKNOWLEDGMENT

This paper is based on research supported jointly by the Kentucky Department of Transportation and the Federal Highway Administration, U.S. Department of Transportation. Findings and opinions expressed herein are ours and not necessarily those of the supporting agencies.

REFERENCES

1. L.E. Peabody and O.K. Normann. Applications of Automatic Recorder Data in Highway Planning. Public Roads, Vol. 21, No. 11, Jan. 1941, pp. 203-222.
2. F.W. Cron. Highway Design for Motor Vehicles--A Historical Review: Part 2, The Beginnings of Traffic Research. Public Roads, Vol. 38, No. 4, March 1975, pp. 163-174.
3. Highway Capacity Manual: Practical Applications of Research. Bureau of Public Roads, U.S. Department of Commerce, 1950.
4. T.M. Matson, W.S. Smith, and F.W. Hurd. Traffic Engineering. McGraw-Hill, New York, 1955.
5. A Policy on Geometric Design of Rural Highways. AASHO, Washington, DC, 1966.
6. A Policy on Design of Urban Highways and Arterial Streets--1973. AASHTO, Washington, DC, 1973.
7. A. Werner and T. Willis. Cost-Effective Level of Service and Design Criteria. TRB, Transportation Research Record 699, 1979, pp. 1-7.
8. W.P. Walker. Trends in the 30th-Hour Factor. HRB, Bull. 167, 1957, pp. 75-83.
9. W.R. Bellis and J.E. Jones. 30th Peak Hour Factor Trend. HRB, Highway Research Record 27, 1963, pp. 1-13.
10. E.F. Reilly and R.D. Radics. 30th Peak Hour Factor Trend. HRB, Highway Research Record 199, 1967, p. 78.
11. B.P. Chu. Michigan's Statewide Traffic Forecasting Model--Volume III: Design Hour Volume Model Development. Michigan Department of State Highways and Transportation, Lansing, Nov. 1972.
12. N. Cameron. Determination of Design Hour Volumes. Department of Civil Engineering, Univ. of Calgary, Calgary, Alberta, May 1975.
13. W.D. Glauz and A.D. St. John. Recreational Travel Impacts--Procedures for Dealing with the Impacts. Midwest Research Institute, Kansas City; U.S. Department of Transportation, Interim Rept., Dec. 1977.
14. ITE Technical Council Committee 6F-2. Reexamination of Design Hour Volume Concepts. ITE Journal, Vol. 48, No. 9, Sept. 1979, pp. 45-49.
15. N.R. DeVries. The Role of Design Hour Volumes in Highway Planning and Design. Wisconsin Department of Transportation, Madison, Working Paper, Nov. 1975.

16. G.E. Maring. Weekend Recreational Travel Patterns. FHWA, Tech. Rept. 18, Feb. 1971.
17. B.G. Hutchinson. Economic Criterion for Highway Capacity Determination. Transportation Engineering Journal, Vol. 98, No. TE3, Aug. 1972, pp. 465-475.
18. Manual on User Benefit Analysis of Highway and Bus-Transit Improvements, 1977. AASHTO, Washington, DC, 1978.

Publication of this paper sponsored by Committee on Highway Capacity and Quality of Service.

Traffic Capacity Through Urban Freeway Work Zones in Texas

CONRAD L. DUDEK AND STEPHEN H. RICHARDS

Findings of capacity studies conducted at urban freeway maintenance and construction work zones in Houston and Dallas are summarized. Studies were conducted on five-, four-, and three-lane freeway sections. The results indicate that the per-lane capacities are affected by the number of lanes open during the roadwork. For example, the average capacity on a three-lane section with two lanes open was 1500 vehicles per hour per lane (vphpl), whereas the average capacity with one lane open was only 1130 vphpl. Also illustrated is how the data can be used to estimate the effects of the lane closure. The results of the study can be used in scheduling work that involves lane closures on freeways.

Findings of capacity studies conducted at 28 maintenance and construction work zones on freeways in Houston and Dallas are summarized. All these studies were made at sites where one or more traffic lanes were closed. A total of 37 studies were conducted at work zones while the work crew was at the site; 4 studies were conducted while the work crew was either not at the site or not occupying a closed lane directly adjacent to one of the open lanes.

FREEWAY WORK-ZONE CAPACITY

Capacity with Work Crew at Site

Figure 1 illustrates the range of volumes measured at several work sites while the work crew was at the site. All volumes were measured while queues were formed upstream from the lane closures and thus essentially represent either the capacities of the bottlenecks created by the lane closures or the effects of drivers staring because of the work crew and machinery. Each point in Figure 1 represents the volume observed during one study; therefore, it is easy to view how the data cluster for each lane-closure situation.

The formula (A,B) is used in this paper to identify the various lane-closure situations evaluated: A represents the number of lanes in one direction during normal operations; B is the number of lanes open in one direction through the work zone.

The average capacity for each closure situation studied is shown in the table below. The data show that the average lane capacity for the (3,2) and (4,2) combinations was approximately 1500 vehicles per hour per lane (vphpl).

No. of Lanes		No. of	Avg Capacity	
Normal	Open	Studies	Vehicles per Hour	Vehicles per Hour per Lane
3	1	5	1130	1130
2	1	8	1340	1340
5	2	8	2740	1370
4	2	4	2960	1480
3	2	8	3000	1500
4	3	4	4560	1520

The studies conducted at work sites with (5,2) and (2,1) closure situations indicate significant reductions in capacity (compared with 1500 vphpl). The average capacity for these two situations was approximately 1350 vphpl.

Studies at (3,1) sites revealed an even greater reduction in capacity. The average capacity was found to be only 1130 vphpl.

Figure 2 shows the cumulative distribution of the observed work-zone capacities. The function of Figure 2 is to assist the users in identifying risks in using certain capacity values for a given lane-closure situation to estimate the effects of the lane closures (e.g., queue lengths).

For example, the 85th percentile for the (3,1) situation is 1020 vphpl. This means that 85 percent of the studies conducted on three-lane freeway sections with one lane open through the work zone resulted in capacity flows equal to or greater than 1020 vphpl. The capacity flow was equal to or greater than 1330 vphpl in only 20 percent of the cases studied. Thus, to assume a capacity of 1500 vphpl for (3,1) work zones would tend to underestimate the length of queues caused by the lane reduction at the vast majority of these work zones.

Because of the limited amount of data, no attempt was made to statistically correlate capacity to the type of road work. There are characteristics at each work site that affect the flow through the work zone. Presence of on ramps and off ramps, grades, alignment, percentage of trucks, etc., also affect the flow. These factors were not evaluated in the studies performed as part of this research.

It is also interesting to note that, even at the same site, there were variations in maximum flow rate. Work activities (e.g., personnel adjacent to an open traffic lane and trucks moving into and out of the closed lanes) caused these variations.

Table 1 is an attempt to summarize typical capacities observed in California by Kermode and Myra (1) and those observed in Texas by the Texas Transportation Institute. The California data represent expanded hourly flow rates, whereas most of the Texas data are full-hour counts. The reader is cautioned that the typical capacities by type of work zone shown in Table 1 for Texas freeways are based