*EVALUATION OF
MAJOR URBAN INTERSECTIONS
DECEMBER, 1960*

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Research
Project*

*PURDUE UNIVERSITY
LAFAYETTE INDIANA*

by

W.W. SCHENLER

Final Report

EVALUATION OF MAJOR URBAN INTERSECTIONS

TO: K. B. Woods, Director
Joint Highway Research Project

December 14, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: C-4-20
Project: C-36-177

A final report "Evaluation of Major Urban Intersections" by William W. Schenler, Graduate Assistant on our staff, is attached. Mr. Schenler also utilized the report for his Ph.D. thesis. The research was conducted under the direction of Professor H. L. Michael.

The report narrates the development of a method of evaluating the adequacy of urban intersections. Physical condition and traffic-movement ability of the intersection are both evaluated, but major importance is given to the latter with this condition measured by the average delay which vehicles encounter. Average delays for intersections controlled by certain fixed-time signals are also evaluated.

The report is presented to the Board for the record.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

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Final Report

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by

William W. Schonler
Graduate Assistant

Joint Highway Research Project

File No: G-4-20

Project No: C-36-17T

December 14, 1960

Purdue University
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ABSTRACT

Schenler, William Walter. Ph.D., Purdue University,
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A rating system is proposed for major urban intersections, which are in reality the source of trouble and delay in urban street networks. The general principles developed apply to all intersections; in addition specific rating values are developed for certain at-grade intersections which are controlled by fixed-time signals. The Intersection Rating proposed is a "sufficiency rating" and is stated as a percentage of a "fully-sufficient" intersection. Factors influencing the ability of the intersection to serve traffic are divided into two general categories which are rated independently and combined to establish the Intersection Rating.

The Physical Rating of the intersection includes ratings for factors which are concerned with the structural and geometric conditions existing at the intersection. Elements rated include surface condition, surface thickness, ridability, skid resistance, geometrics, curb radius for right turns, visual restriction and lighting, among others. These factors rate the intersection as a physical unit, without regard for the amount or characteristics of traffic or control involved, except that general traffic volume levels do influence certain

of the ratings such as surface thickness and lighting.

The Traffic Rating is a device for evaluating user satisfaction with the service rendered by the intersection under existing volume and control conditions. Average delay per vehicle was selected as the criterion, and a rating scale was established. For the restricted case where uniform arrival and fixed-time signals exist or can be assumed, theoretical volume-delay relations were established for appropriate cycle lengths and cycle divisions. These relations are presented in graphical form. From these volume-delay relations, Traffic Rating graphs were prepared. These graphs also cover the cycle lengths and divisions considered appropriate. The derivation of the rating graphs is explained.

In order to determine whether or not there was reasonable agreement between the theory and actual traffic flow, field investigation was undertaken. The procedures used and results obtained are discussed in connection with the Traffic Rating. It was concluded that there was reasonably good agreement between the theory and actual traffic performance.

Since the Traffic Rating is of necessity based on the individual approaches, and the Physical Rating represents the intersection as a unit, it was necessary to get to a common basis. A composite Traffic Rating for the intersection as a unit was calculated; then, with both ratings on an intersection basis, they are suitable for combining to get the Intersection Rating. Certain boundary conditions were

considered appropriate and desirable; these are discussed in some detail. A suitable combining relation was developed, and a table is presented for convenient solution of this equation.

Following the completion of the rating system it was again necessary to investigate in the field, to see whether or not results of the proposed rating agreed with ratings based on informed opinion. The comparison of opinion ratings for selected intersections (made before application of the formal rating system) and corresponding Intersection Ratings is shown and the validation study discussed.

INTRODUCTION

By any criterion, the national "highway problem" taxes the individual's ability to comprehend its magnitude, whether the problem be stated in terms of numbers of vehicles to be accommodated, mileage of roads and streets in service, or annual costs involved in new construction and maintenance of existing facilities. Since New York State pioneered motor vehicle registration in 1901, the total registration has climbed from 14,800 (1901) to 71,502,400 in 1959. One million registrations were recorded in 1913, it taking twelve years to reach this mark. Two years later in 1915, the two-million mark was reached, and three million registrations were recorded the following year. For the next thirty years until 1947, registration increased at a rate of one and one-half to two and one-half million vehicles per year, excepting the depression and war years. Since 1947, the increase has been of the order of two and one-half to four million vehicles annually (32, 34, 58).

Correspondingly, the road net to serve these vehicles developed rapidly (on a mileage basis), then has remained reasonably constant over a period of years. Latest available figures (calendar year 1958) indicate the total U. S. road mileage to be approximately 3,478,000 miles, of which 404,600 miles are classed as urban mileage, the remainder being rural

mileage (34).

A cost estimate for such a system is difficult if not impossible to establish, but records indicate that from 1921 through 1959, capital outlay for highway purposes at all levels of government was approximately seventy-six and one-half billion dollars (31, 32, 33, 34). From 1955 to 1958 U. S. highway expenditures increased nearly forty per cent (73). Mr. Bertram Tallamy, current Federal Highway Administrator, recently pointed out that in the three and one-half years from July 1, 1956 to December 31, 1959, federal-aid highway projects totalling \$7.5 billion were completed on 100,000 miles of roads - more money than was spent by the federal-aid program from its inception in 1916 through 1948, a period of over thirty years! It is further anticipated that 1960 will bring expenditures of nearly \$7 billion, both on and off of the federal-aid system, and vehicular travel of 725 billion miles. By 1971 it is estimated that vehicular travel will exceed one trillion miles, of which 40 per cent will be urban (76). Hence 40% of the traffic will use 10% of the roads.

Need for Rating Systems

Clearly, by any measurement, highways are "big business" and becoming more important as time passes. Consequently, it is of utmost importance that those persons in authority in the highway field recognize a duty to utilize the funds at their disposal to promote the public interest in the best

possible manner - the most good for the most people, on a non-partisan basis (2, 60).

As one method of implementing such a policy, a Highway Sufficiency Rating System was established as a joint effort of the Arizona Highway Department and the Bureau of Public Roads in 1946. Since that time, many states have adopted the basic idea while altering the details of application to suit their individual needs. The basic concept is that every road section is evaluated in accordance with its ability to meet the demands placed upon it, utilizing certain arbitrary standards for comparison purposes. In general, all rating systems attempt to give an evaluation of the ability of each road section to carry traffic safely, rapidly and economically (1, 42).

It should be noted that some states have investigated sufficiency ratings and rejected them, feeling there was not a true measure of relative sufficiency (1, 42, 70). However, it is contended by others that the relative weighting of the categories is not too important, and that what is important is that the rating be done on a uniform, impartial, engineering basis. It has also been pointed out that the results of such investigations are not to be used blindly, but are to be used in conjunction with other factors as a guide to priority of reconstruction (1, 39, 64, 70, 78).

It is not the purpose here to become involved in a discussion of the advantages and/or disadvantages of such rating

systems. Both the advantages and disadvantages are concisely set forth in the literature (1, 60, 74). The fact remains that more and more states are adopting some variation of the basic sufficiency procedures, an indication that such systems must be reasonably satisfactory (1).

Coverage of Existing Systems

The original "Arizona" system was devised for application on the rural primary highway system, and most present variations are also thus used. If a state uses a rating system at all, it is certain to be for its rural primary roads. Some states, however, are now expanding their coverage. A rating for local rural roads has been recommended for Indiana (2), and other states have also considered the problem of rural roads on other than the primary system (71).

Ratings of urban facilities are becoming more common. Here the procedure is usually an adaptation of the method used to rate the rural primary roads, and only those facilities are rated which are urban extensions of state roads (19, 56, 62). To quote Curtis J. Hooper, Conn. Highway Department:

"Across the nation the state highway departments have always devoted the major part of their effort toward the rural sections of the state. Only in the recent past have their obligations been broadened to include the problems found on arterial streets in the incorporated communities.

"Having devoted a great number of years to the elimination of 'rural mud,' we are now faced with the obligation to do something about the 'urban muddle'" (36).

However, a few cities, in this country and elsewhere, have proceeded on their own to study the arterial portions of their street systems with ratings of various kinds (38, 44).

In addition to the various "sufficiency ratings," other concepts have recently been suggested. A "congestion index" has been proposed for both rural and urban applications (36, 64, 65). A frictional concept of traffic flow has been suggested (53), and quality of traffic transmission has been discussed (25). The literature also reveals a comparison of several such evaluation methods which concludes "no common ground can be found for comparing such widely differing values" (14).

It should be noted that the bibliography references cited in connection with the various rating systems should be considered as typical, but are by no means an exhaustive review of such sufficiency reports and literature as are available in these fields.

Need for a New Rating Device

Considering the rating systems just mentioned, it appears that at least one more is required to make the coverage of all parts of the highway system more comprehensive. Thus far there is no method whereby urban intersections may be examined and compared on an over-all basis. It is widely recognized that intersections are the chief factor contributing to traffic strangulation on traffic facilities, and a method

of intersection evaluation should be a valuable addition to the toolkit of the highway and traffic engineers and planners (1, 30, 40, 64, 77). This report describes the development and use of such an Urban Intersection Rating.

A suitable intersection rating should accomplish the following:

1. Obtain an unbiased estimate of the ability of any intersection to meet both the traffic and structural demands placed upon it;
2. Offer a method of comparison of various intersections to aid in the establishment of priorities for improvement;
3. Evaluate alternate methods proposed for the improvement of any intersection; and
4. Provide an over-all measure of system adequacy, since urban facilities are mainly governed by the characteristics of their intersections. Thus improvement in system adequacy indicates progress made in dealing with intersection problems, and conversely.

Accomplishment of number 4 above requires that the evaluation be repeated periodically, to see whether progress is being made or ground is being lost in the effort to provide better facilities (1, 39, 78).

It is appropriate at this point to restate certain characteristics that any rating system should possess. Concisely stated, if the system is to have any value it must have validity (that is, the answers by rating formula must agree

with expert engineering judgment), and discrimination (that is, the ability to distinguish between facilities which are nearly identical). The system should also be objective in its application. From a practical standpoint, the rating should be simple in nature and economical to accomplish. Universality in application would also be desirable, but is not so necessary as the above items (1, 2, 42, 70, 78).

Selection of General Rating Factors

It was considered that all factors which affect the ability of an intersection to accommodate the demands placed upon it could be placed in one of two general classifications, namely the physical factors and the traffic factors.

Selection of the appropriate physical factors is discussed in the second section of this report, and objective methods of evaluating these factors are established. Mention is also made of certain factors which were considered but not adopted for inclusion. Relative weights are assigned each factor, and the "Physical Rating" is obtained by summing the point ratings awarded the various factors.

The third section is concerned first of all with determining a criterion of user satisfaction or dissatisfaction with intersection traffic conditions. Then the relation of appropriate measurable characteristics, such as traffic volume, calculated approach capacity and certain other factors of importance to the criterion is established for the particular case where control is by fixed-time signals. Following the

establishment of this relationship, derivation of a "Traffic Rating" is accomplished. Determination of such a relation was necessary to be able to predict quality of traffic flow for projected intersection alterations, which was previously mentioned as one of the reasons an intersection rating procedure is needed.

The fourth section of the report combines the "Physical Rating" and the "Traffic Rating" in accordance with certain boundary principles to give the "Intersection Rating."

Accidents Excluded

It will be noted that accidents, as such, have not been included as a factor under either of the two general categories. This was done for two reasons: first, it is widely believed that accident data are so incomplete as to render results based thereon highly questionable; second, the available data are to a high degree inaccurate for various reasons (1, 5, 11, 21). In view of this general questioning of such data, the accident factor was omitted, but it was recognized that safety, or the lack of it, is inherent in many, if not all, of the factors which were considered, and was thus included indirectly (56).

It was specifically recognized that accidents are symptomatic of operational deficiency, hence valuable in isolation of high-accident-frequency locations for special analysis and treatment (5, 30). However, studies indicate that accident prediction based on roadway features and roadside ele-

ments is not successful within reasonable limits (29, 41, 54, 67).

"Emotions and attitudes are not among the reported causes of accidents. As a result, there is no accurate information about the number of accidents they cause" - Bruce Greenshields (24). Libert Ehrman concurs (21).

James S. Burch said "- we also believe that real progress in accident reduction is primarily a matter of driver training, driver examination and enforcement of safe driving laws combined with motor vehicle inspection" (11).

There were too many intangibles associated with accidents and accident records to include such a factor in a rating system directly.

FIGURE 1. EXAMPLE OF A MAJOR URBAN INTERSECTION.

THE PHYSICAL RATING

The factors adopted for inclusion in the Physical Rating were, in general, those concerned with the estimated life of the pavement from a structural standpoint and with the geometric layout of the intersection. Certain other minor factors relating to intersection details were also included.

The factors used, and the assigned weights, are as follows:

Table 1

Physical Rating Factors

<u>Factor</u>	<u>Points</u>
Surface condition	20
Ridability	5
Skid resistance	5
Intersection geometrics	20
Curb radius for right turn	5
Visual restriction	5
Lighting	5
Other pertinent characteristics	<u>5</u>
	70

Detailed analysis and evaluation procedures for the above factors will be found in the paragraphs to follow.

Surface Condition

The Surface Condition rating used effectively rates all factors affecting the structural strength and longevity of the pavement as a physical structure. Functional obsolescence (such as inability of an intersection to accommodate the volume of flow) is covered in later sections. The procedure utilized takes the surface condition of the pavement as indicative of the condition of the entire pavement structure on the basis that failure of any portion of the structure will be reflected or indicated by corresponding surface distress. Pavement evaluation is thus on a performance basis. Rating is simplified, yet the requirement for an objective method is met. This general approach is used in the 1960 Sufficiency Study Manual of the State Highway Department of Indiana for rating roadways for structural adequacy.

The rating is based on field evaluation of the "maintenance requirement" of the intersection pavement. In making the evaluation all evidence of existing or impending failure is considered, including pumping, faulting, warping, map cracking, raveling, creeping, spalling, scaling, frost heave, failure of bituminous patching and resurfacing, wash-boarding, chuckholes, extruded joint filler, etc. Separate rating scales are provided for Portland cement concrete and bituminous surfaces. These scales cover maintenance measures appropriate to the respective surface types, and require reduction in the Surface Condition rating as the maintenance measures found

necessary increase in scope and severity. Table 2 lists the possible maintenance measures associated with Portland cement concrete surfaces along with rating ranges considered appropriate for surfaces with these maintenance requirements; Table 3 does likewise for bituminous surfaces. It will be noted that appropriate maintenance measures vary in some respects between the two surface types, recognizing the fundamental structural difference which exists between rigid and flexible pavements.

The surface conditions associated with the various maintenance requirements are listed in Table 4 for Portland cement concrete surfaces and in Table 5 for bituminous surfaces. Assignment of the actual point rating within the appropriate maintenance-requirement classification depends on the relative need for maintenance and must necessarily be left to the judgment of the evaluator.

Ridability

An evaluation of the smoothness of the ride afforded by a surface should enter into an Intersection Rating. This factor could conceivably enter through the Traffic Rating, but appears more logically placed as a Physical Rating factor. It is not one of the values considered in the customary capacity calculations; furthermore it seems doubtful that surface roughness affects intersection speeds or capacities except in extreme cases. For these reasons, plus the fact that roughness is a direct function of materials used and construc-

Table 2

Surface Condition Ratings For Portland Cement
Concrete Surfaces

<u>Maintenance Requirements</u>	<u>Rating Range</u>
No Maintenance Required	18-20
Joint Sealing Only	14-17
Patching	7-13
Resurfacing and/or Undersealing	3-6
Complete Reconstruction	0-2

Table 3

Surface Condition Ratings For Bituminous Surfaces

<u>Maintenance Requirements</u>	<u>Rating Range</u>
No Maintenance Required	19-20
Minor Patching and/or Overlay Joint Maintenance	15-18
Moderate Patching	11-14
Extensive Patching and/or Sealing	7-10
Resurfacing	3-6
Complete Reconstruction	0-2

Table 4

Maintenance Requirements For Portland Cement Concrete Surfaces

<u>Maintenance Requirements</u>	<u>Deficiencies</u>
No Maintenance Required	No evidence of displacement: no pumping, faulting, vertical alignment problems, etc. Slabs comparable to new in riding quality, but may show evidence of cracking on up to 5 percent of surface area. All edges of cracks are sharp and no evidence of spalling, scaling, or raveling of aggregates can be found.
Joint Maintenance	No marked evidence of joint displacement and faulting. Only slight vertical alignment problems noticeable in some sections. Usual joint maintenance and sealing procedures are adequate. Slight spalling at cracks.
Patching	Very minor subgrade support problems. Surface rough in some sections. Joints and cracks spalled and/or faulted, needing localized patching.
Resurfacing and/or Undersealing	Subgrade support problems, pumping and faulting. Rough surface over all of the section. Aggregate exposed and raveled out in some sections. 50-75 percent of slabs show severe cracking.
Reconstruction	Extremely poor riding quality. Evidence of extreme slab movement on passage of vehicles. Surface pitted and aggregate raveled out from scaled sections. 75-100 percent of slabs show severe cracking.

Table 5

Maintenance Requirements For Bituminous Surfaces

<u>Maintenance Requirements</u>	<u>Deficiencies</u>
No Maintenance Required	No evidence of deformation or cracking. Smooth surface except for normal shrinkage cracking.
Minor Patching and/or Overlay	Very slight deformation and cracking. Less than 10 percent of surface area needs patching.
Joint Maintenance	
Moderate Patching	Slight deformation and cracking. Between 10-50 percent of surface is in need of patching.
Extensive Patching and/or Sealing	Slight deformation and moderate map cracking. About 50 percent of surface in section is breaking up in various stages.
Resurfacing	Considerable deformation and severe map cracking with most of the surface showing evidence of some break-up.
Reconstruction	Clear evidence of base and/or subgrade failure. Extremely distorted and broken surface.

tion methods employed, ridability was included in the Physical Rating.

An objective method of rating ridability was desired. Frank M. Holloway in a paper "Road Roughness Measurements of Indiana Pavements" (35) describes a "roughometer" which is essentially the same as the "Relative Road Roughness Indicator" used by the Bureau of Public Roads, and which has been used in roughness measurements of certain Indiana roads.

Emphasis is placed on the word "relative" in the BPR "roughometer" title. For tests to be comparable between runs with a given BPR - type machine, or between two or more such machines, certain standard conditions must exist or the "sprung" characteristics of the devices change. Standard Bureau of Public Roads operating conditions are 20 mph $\pm$ 1/2 mph and 30 psi tire pressure $\pm$ 1/2 psi for all machines built to their specifications, so that results may be comparable.

The stated speed of 20 mph is suitable for intersection evaluation. However, the machine itself is not designed for the measurement of short distances or small increments of surface roughness. Present equipment has a least count of one inch on the vertical roughness integrator and one wheel revolution for distance. Refined equipment, as proposed in the Holloway paper, or something analogous, will be required before intersection roughness may be measured directly.

Holloway's work does, however, provide an indirect, but still objective, method of evaluation. For various types

of pavement as constructed in Indiana roughness values were obtained for good, average, and poor road sections. Tabulation of his results, with point scores for this rating added, are shown in Table 6.

It should be noted that the above table as pertains to pavement types is based on current conditions as found in Indiana. Holloway cites work in other states to the effect that Portland cement concrete pavements, for instance, can be, and are being, built with inches-per-mile roughnesses as low as 38. If performance specifications are tightened, and enforced, in Indiana, the above table should be revised.

Holloway also compared engineers' ratings of roughness based on their opinions after traversing certain sections with roughometer results. He found personal opinion to be highly variable concerning riding quality of the road sections, and concluded that the opinion method is therefore an unsatisfactory evaluation method. This again emphasizes the desirability of an objective rating method.

It would be preferable to directly evaluate pavement roughness through the intersection, of course, but the present state of the art does not permit it. The approach presented here appears to be the most logical alternate procedure.

Skid Resistance

This factor is a function of the materials used in pavement construction, and is included because of its effect on vehicle stopping distance when the surface is wet. It is

Table 6
Ridability Rating

Ridability Description	Roughness Inches per Mile	Pavement Types					Points
		Silica Sand	Rock Asphalt	Portland Cement Concrete	Bituminous Concrete	Bituminous Coated Aggregate	Bituminous Surface Treatment
Excellent	Below 60		Best		Best		5
Good	60-75	Avg	Avg	Best	Avg+	Best	4
Fair	75-90		Worst	Avg	Avg-	Avg+	3
Poor	90-150			Worst	Worst	Avg-	2
Very Poor	Above 150					Worst	1

essentially a measure of relative safety. The wet condition is of importance because for practical purposes all pavements when dry produce adequate and reasonably similar stopping distances, but when the surfaces are thoroughly wetted, stopping distances increase and become more variable.

To objectively evaluate stopping characteristics under such conditions, reference is made to the work of David L. Grunau concerning skidding properties of the various surfaces as constructed in Indiana (27). Grunau's work resulted in obtaining average stopping distances ("S") for different surface types from a standard testing speed of 30 miles per hour. In Table 7, the various surfaces were arranged in order of their mean skid stopping distances, and rating points assigned. Under other test conditions it is reasonable to expect that different values would be obtained for the mean stopping distances; however the relative order of the surfaces as ranked according to such stopping distances is expected to remain the same.

The following assumptions are inherent in the use of the actual stopping distances determined by Grunau:

1. 30 miles per hour is a reasonable speed for the evaluation of skid resistance at intersections;
2. Changes in "S" due to variation of tread type, tire pressure, amount of wear and vehicle weight will be of negligible effect for the purposes here intended; and
3. Test results will be sufficiently reproducible among vehicles that the "S" values determined by test will reasonably

Table 7

Skid Resistance Rating

Stopping Distance "S"	Distance Increment ΔS	Skid Resistance Rating	Surface Type and Kind of Coarse Aggregate						
			Rock Asphalt	Stone	Portland Cem. Conc.	Gravel	Concrete	Bituminous Surface	Gravel
60	10	5	Avg						Best
70	10	4		Best Avg+		Best Avg+			Avg+ Best
80	10	3		Avg- Worst		Avg- Worst	Best Avg		Avg- Avg+
90	10	2					Worst Avg Worst		Worst Avg-
100	10+	0							Worst
110+									

Note: Word entries in table are descriptive of surface skid characteristics.

represent all typical passenger vehicles.

In establishing Table 7, it was recognized that different sections of streets with the same nominal surface type have different stopping distances. For the given surface types, surfaces rated as being in "average" condition are those which had the average stopping distances reported by Grunau. Surfaces with "best" or "worst" stopping-distance characteristics were entered as appropriate.

It must be emphasized that these ratings are based upon tests run on Indiana highways, as constructed with locally-available, customarily-used materials. If other materials or construction procedures are used, as in other states, these relative results should be verified before using Table 7.

As in the preceding section, this procedure was adopted as the most reasonable alternate to the theoretically-desirable but practically-impossible task of testing each intersection to be rated.

If different intersection approaches have surfaces of varying types, the rating for the approach with the poorest skid characteristics is used for the intersection rating for this factor.

Intersection Geometrics

This section is the first of two which deal with the geometric configuration of the intersection, or parts of it. The concern here was one of safety in operation through proper geometric design. The principal way such operation

may be promoted is by restriction of vehicles to predictable paths, which may be accomplished in either of two ways. First, and more commonly in the urban street network, the physical size of the intersection is so small that there is little opportunity for erratic or unpredictable behavior on the part of a driver. The intersection thus does its own restricting. In other intersections the gross area is large because of the number of approach streets, intersection angles, offset centerlines or other such reasons. These large areas present opportunity for "broken field running," and unpredictable driver actions in general. Channelization serves in such cases to improve the situation by restriction of vehicular movement to predetermined paths and is the second method of vehicle path restriction referred to above.

An objective method of approaching this problem must be capable of dealing with the extreme cases as well as those intermediate in nature.

The general approach utilized is as follows. First the area which constitutes an intersection is established, by definition and then by measurement. Second, the actual area is compared to a theoretical calculated area called the "standard area" and an "area ratio" and basic score are obtained. If no channelization exists, this score is the final point rating for the Intersection Geometrics factor.

If channelization does exist, the basic score as determined above is adjusted by addition of a correction, which

is found by determining the length of channelized paths in the intersection and dividing by the total length of all these paths. The magnitude of this ratio is used to determine the value of the correction. The sum of the basic score and the correction may not exceed 20 points.

After the corrected score is obtained, further evaluation is made to take into account the quality of the channelization. The evaluation is accomplished by deducting points if certain conditions are found to exist which are not in accordance with usually-accepted standards of good practice. This "quality evaluation" must be made if a channelization correction is applied as discussed in the preceding paragraph.

The following paragraphs will enlarge upon and explain the procedures just summarized.

Actual Area

For this evaluation, the intersection is considered to begin where the straight curb line connects with the curved curb which outlines the corner. On each approach a line is drawn across the approach, connecting these points of tangency on opposite sides of the street. All such lines, plus the curved curb lines, bound the intersection area. Three-way intersections represent a special case. In such cases boundary lines are drawn across the "through" approaches parallel to the centerline of the side street to define the intersection area. This area is the gross area, without deductions for any islands or other non-paved area within the

boundary thus established. See the typical examples in Figure 2. The actual number of square feet is then obtained in any convenient manner.

Standard Area

The "standard area" for an intersection may not be calculated until certain dimensions of the actual intersection are known, because the "standard area" is tailor-made to correspond to the field intersection in every case.

The "standard area" has the following characteristics:

1. The angle of intersection is 90 degrees;
2. The width of the principal street (w_p) is the average curb to curb width* of the main street at the actual intersection (entering and leaving widths averaged);
3. The width of the minor street (w_m) is the average curb to curb width* of the minor street of the actual intersection (entering and leaving widths averaged);
4. The uniform corner radius (R) is the average of all the radii of the existing intersection;
5. The intersection is bounded by the lines and arcs previously described for actual areas.

*If either street has a center median, do not include the median width in the curb to curb width.

Figure 3 shows these characteristics graphically.

In the case of a five-way (or more) intersection use the width measurements from the two major through streets to determine the standard intersection. Average all corner

FIGURE 2. TYPICAL "ACTUAL AREAS."

FIGURE 3. STANDARD AREAS.

radii to evaluate R.

For tee or branch intersections, use the side road as the minor road and average all radii. Note that the special "standard area" (Case II, Figure 3) applies to three-way intersections.

If any individual corner radius is longer than eighty feet, then eighty feet shall be used in place of the actual radius value when computing the average radius R as referred to in this section.

Figure 4 is a graphical solution for the "standard area," given W_P and W_M (see Figure 3), R and a Case I intersection. Entering the graph with W_P and W_M , the gross area is determined on the right leg of the "N" part of the graph by laying a straightedge through W_P and W_M and reading the gross area A_G . By using the gross area value as a pivot value and reading A_G and R under the straightedge the net area value A_N is found where the straightedge crosses the net area line. Figure 5 is similarly used to determine the "standard area" of a Case II (three-way) intersection.

Area Ratio - The Basic Geometric Rating

The actual and standard areas just obtained are then combined to determine the "area ratio" by dividing the "standard area" by the "actual area," thus:

$$\text{area ratio} = \frac{\text{standard area}}{\text{actual area}}$$

This form was adopted because the ratio value must lie

$$A_N = (w_p + 2R = w_p)(w_m + 2R = w_m) - \pi R^2$$

MINOR WIDTH W_M (FEET)

GROSS AREA A_g (THOUSAND SQ. FT.)	NET AREA A_N (THOUSAND SQ. FT.)	AVERAGE CURB RADIUS R (FEET)
34	14	50
32	30	40
30	24	30
28	24	20
26	24	10
24	22	0
22	20	
20	18	
18	16	
16	14	
15	12	
14	10	
12	8	
10	6	
8	4	
6	2	
5	0	
4		
2		
0		

FIGURE 4. DETERMINATION OF STANDARD AREA FOR FOUR (OR MORE) WAY INTERSECTIONS. CASE I.

FIGURE 5. DETERMINATION OF STANDARD AREA FOR THREE - WAY INTERSECTIONS. CASE II.

between zero and unity. For given street widths and corner radii, the area of the standard intersection is the smallest area obtainable. Deviation of any intersection condition from ideal increases the actual area of the intersection, and the ratio decreases from unity toward zero.

It was considered that minimum actual area (that is, actual area equals the standard area) should rate a basic score of 18 out of the maximum 20, considering area concepts alone. It was felt that even intersections of minimum area could benefit at least slightly by minimum channelization, so full score was not attainable without the channelization correction being applied. This correction is more fully discussed below.

The area ratio is related to the basic score by the following equation:

$$S = \frac{R - 0.63}{2}$$

where R is the area ratio and S is the basic score. These results are graphed in Figure 6 and tabulated in Table 8.

Table 8

Relation of Basic Score to Area Ratio

<u>Area Ratio</u>	<u>Basic Score</u>	<u>Area Ratio</u>	<u>Basic Score</u>
.980 - 1.000	18	.800 - .819	9
.960 - .979	17	.780 - .799	8
.940 - .959	16	.760 - .779	7
.920 - .939	15	.740 - .759	6
.900 - .919	14	.720 - .739	5
.880 - .899	13	.700 - .719	4
.860 - .879	12	.680 - .699	3
.840 - .859	11	.660 - .679	2
.820 - .839	10	.640 - .659	1
		Below .640	0

FIGURE 6. INTERSECTION GEOMETRICS BASIC SCORE.

Let it be emphasized again that widths of median sections are excluded from curb-to-curb widths in calculating standard areas, but actual areas include any areas of islands or medians which are present within the intersection area. Counting these latter areas as paved area results in a lowered basic score because the actual area is inflated. Correction for this is made in the channelization-correction procedure.

If no channelization exists, the basic score becomes the final point rating for the Intersection Geometrics factor.

Channelization Corrections

If channelization islands and/or medians exist at the intersection, these should act to reduce or eliminate the harmful effects of excess area. After considering several methods of applying a correction to the basic score to account for this effect, the method described below was adopted as most suitable to the wide range of conditions required to be covered.

The correction is calculated by measuring the lengths of all possible paths through the intersection, and obtaining the total of these distances. Only one path is used for each directional movement through the intersection, and that path most commonly used for each movement is chosen. Then each such path is examined again and the length of the path which is laterally restricted, or channelized, is measured. The total of these channelized distances is obtained. Then the correction is figured by the equation

$$C = 20 \times \frac{\text{Total of channelized lengths}}{\text{Total length of all intersection paths}}$$

In determining channelized path lengths, a path is considered channelized if all of the following conditions are met:

1. The path has the same number of lanes as at the intersection entry, or fewer;
2. The lane width on curved sections increases no more than 5 feet above the width of the lane on tangent alignment; and
3. Side restrictions as defined below exist along both sides of the path.

The following physical conditions are considered to produce "restriction" within the meaning established above:

1. Pavement edges,
2. Curbs of all kinds,
3. "Jiggle bars,"
4. Traffic buttons,
5. Islands,
6. Painted island extensions,
7. Lane lines setting off separate turning lanes, and
8. Any other physical dividers or separators.

The following conditions are considered as producing no side restriction:

1. Painted lane and centerlines,
2. Painted islands,
3. Signs, and
4. Painted legends and arrows on pavement.

Where channelized paths cross, obviously side restrictions must be interrupted for short distances. In determining restricted lengths, such short distances (say 40 to 50 feet) are ignored and the channelized length measurements carried through such gaps continuously. Use of judgment is required in this connection.

From the correction formula above it will be noted that the maximum value of the correction is 20 points. This correction is added to the basic score as determined from area considerations. Thus an intersection with channelization, penalized on the basic score by using gross area as previously noted, can regain as much as 20 points if highly channelized. To provide a reasonable and suitable rating the basic score plus the correction may not exceed 20 points.

Thus far, no rating element has been introduced to discriminate with regard to the quality of the channelization. Obviously, no channelized intersection should get top rating because of the magnitude of channelization provided, irrespective of quality. Consequently, from the adjusted score (maximum value of 20, as noted above) certain deductions are now made if certain conditions are found to exist which are contrary to best available expert opinion. Each deduction is made only once for an intersection, even if the fault occurs more than once.

Table 9

Deductions for Poor Channelization Qualities

<u>Fault</u>	<u>Deduction</u>
Undue distortion of a major flow path	2
Undue distortion of a minor flow path (over and above the usual rule of "bend the minor flow")	1
Islands not offset at least 2 feet from the edge of a traffic lane	1
Crossing movements not near 90° angles	2
Merging movements not at flat (10°-15°) angles	2
Funneling not used where feasible	1
Shadowing not used to protect waiting or crossing vehicles where feasible	1

Summary

Any large unchannelized intersection is penalized by this rating system. This includes staggered intersections and also those which are skewed, because the area of a skewed intersection varies almost directly with the cosecant of the angle of intersection. Intersections with five or more legs are penalized because they are compared to a "standard area" with only four legs. The greater the number of legs, the worse the area ratio and the greater the penalty.

Intersections without channelization and with near-minimum area benefit little from channelization. It was previously mentioned, however, that the maximum rating for such an intersection is set at 18 out of 20, it being considered that short centerline islands on each approach benefit

even this near-optimum case.

Unchannelized intersections with excess area may have their ratings improved by channelizing. Good design avoids the stipulated deductions and maintains a high adjusted rating.

Similarly, projected improvements in channelization may be evaluated.

A well-channelized intersection should receive the maximum rating of 20, or very close to it, regardless of the basic score.

Remarks

While the principles of channelization appear to have been rather definitely established (13, 47, 48, 49, 61), there have been no warrants devised to set forth the point at which channelization becomes desirable (13, 50). In fact, some experts in the field have expressed doubt that such warrants are feasible and/or desirable (13, 61). Perhaps E. T. Perkins summed it up when he said,

"Typical examples' are used as guides - channelization design at present is more of an art than a science." (61)

If the decision is made to channelize, guidance as to minimum island size, set-back and curb types may be found in the design policies of the American Association of State Highway Officials (23, 37).

Channelization as contemplated in this report is localized at the intersection. However, for an interesting ex-

pansion to separate adjacent and multiple roads, reference is made to the report of W. R. Bellis on certain work in New Jersey (8).

Curb Radius for Right Turn

For an intersection to be geometrically adequate, it is necessary for all vehicles to be able to execute turning movements without unduly interfering with other traffic (22). Because of the longer radius, left turns are usually not critical in this respect except in rare cases, so this discussion is restricted to right turns.

Passenger-car turning movements are very seldom a problem, since a car can negotiate any reasonable radius with little or no difficulty. Truck turns are the essence of the problem, and the first step in any solution is to determine for what type of truck a design should be made in any given case. For instance, how often must trucks of the C-50 design class turn a corner to justify design for this standard vehicle, over and above the design required for the SU or C-43 design vehicles? Any answer is arbitrary, as there is no known research on this subject. The standard selected here was that design for a vehicle class is justified and desirable if in any hour twelve such vehicles make the turn.

The problem is further complicated by two other factors. First, various turning maneuvers are possible, depending on the approach being considered. The turn may be made from curb lane to curb lane where parking is not allowed (at least

near the corner), or from the curb lane to any exit lane. In the latter case, crossing of the centerline and infringement on the approach lanes of that leg may or may not occur. Or, turns may be made from second lane to second lane where parking is permitted very close to the corner on both approach and exit. In this connection, the American Association of State Highway Officials says:

"Caution in the use of radii of 15 or 25 feet is advised even where parking can be restricted at first, because if volumes increase to the point where all parking is prohibited, either during rush hours or throughout the day, the same conditions will obtain as in Figure E-4 and E-5" (turning movements with no approach parking) (46).

The second complicating factor is that the path of the selected design vehicle is highly dependent upon the angle of turn.

Considering the factors just mentioned, the following rating method was established. The selected design vehicle was assumed to approach the intersection in the curb lane, and not to encroach on any other approach lane. From Figure 7, the maximum width of exit required for the turn was determined when the angle of turn and curb radius R were known. Then the exit width required was compared to the exit width available. If no encroachment into the intersection-approach lanes of the exit roadway was required, the maximum rating of five points was awarded. For each foot of encroachment, one rating point was deducted.

The worst rating obtained on any of the intersection

FIGURE 7. REQUIRED EXIT WIDTHS.

approaches was used as the Curb Radius for Right Turn rating for the intersection.

Visual Restriction

In considering the factors of the Physical Rating, a need was felt for one which deals with visibility, or the lack of it. This factor is not to be confused with illumination, which will be discussed next; it deals with vision restriction because of obstacles rather than because of darkness.

At controlled intersections, the driver (theoretically, at least) does not need to worry about what the cross traffic is doing. Nevertheless, it seems certain that inability to observe the potential cross-interference and danger leads to a sense of restriction, and a probable reduction in capacity. It would be logical, therefore, to deal with visual restriction as yet another factor of intersection capacity. However, no such factor has thus far been isolated in any of the currently-common methods of calculating intersection capacity.

Hence, the logic of treating visual restriction as a physical characteristic of the intersection was considered. Since the effect is really a combination of vehicular and physical intersection characteristics, it may be dealt with under either category with good justification. It was included here under the Physical Rating so that the standard capacity calculations could be used without alteration.

The evaluation method chosen here is based on the

American Automobile Association's graphical method of determining safe approach speeds at intersections. Other similar procedures are available, and modification of any of them would probably give equally satisfactory results (69).

In brief, the original AAA method determined the safe approach speed on the cross street, given major street vehicle speed, obstruction location and vehicles in the worst possible legal position. The modification used here says that if conditions are such that the safe cross-street speed is 25 miles per hour, there is in fact no restriction to urban operation, and such a condition should be worth the full five-point rating.

Figure 8 shows how the critical view obstruction is located with respect to major and minor street vehicles in their most dangerous legal positions. With the view obstruction thus located, Figure 9 is used to determine the Visual Restriction rating.

Detailed instructions and procedures for using Figures 8 and 9 are as follows:

1. Determine the values of a' and b' . For most dangerous legal position, the value for a' is either 12 feet (with parking) or 6 feet (without parking); the value for b' is either one-half the street width plus 3 feet, or the street width minus 12 feet, whichever is smaller.
2. Measure distances from view obstructions to curb lines (a'' and b'').

a' AND b' ARE THE DISTANCES
FROM THE DRIVERS TO THE
CURB LINES.

a'' AND b'' ARE THE DISTANCES
FROM THE CRITICAL VIEW
OBSTRUCTION TO THE CURB
LINES.

a AND b ARE THE DISTANCES
FROM THE DRIVERS TO THE
VIEW OBSTRUCTION.

FIGURE 8. CRITICAL VIEW OBSTRUCTION LOCATION.

3. Determine the critical distances a and b ($a = a' + a''$; $b = b' + b''$).

4. Locate the most restrictive view obstruction at the intersection on the chart, Figure 9, by using the values of a and b obtained above.

5. Determine the speed value for the major street by using the value of the speed limit.

6. Draw a straight line through the speed value in miles per hour for the major street (on the "A" scale) and the point of the view obstruction as located in item 4 above.

7. The intersection of this line with the "B" scale is the Visual Restriction rating relative to the view obstruction.

8. This rating for the most restrictive obstruction is used as the intersection rating for this factor.

Lighting

It seems clear that there are a number of factors which are responsible for the increase in the night accident rate as compared to the rate during daylight hours. Of the factors listed by David M. Baldwin (4), the only one the engineer can do much about at present is the reduced visibility. Baldwin states:

"The best evidence that the change in visibility contributes to increased night accident experience lies in the many examples of accident reduction following efforts to increase night illumination. Street lighting improvements have almost universally been followed by fewer accidents and deaths. Perhaps even drunken drivers are better drivers when

they can see where they are going.

"Certainly the most direct and obvious approach is that of providing improved street lighting."

Technical Committee 4D of the Institute of Traffic Engineers has amply reinforced the above quotation with a report covering results in numerous cities across the country (20).

The best available guide to acceptable lighting practice is the table of minimum values of illumination levels, developed by the Illuminating Engineering Society and adopted by the American Standards Association. Table 10 of this report is based on an IES table, but is expanded to evaluate directly surfaces with all three of the commonly-specified reflectances (3%, 10%, and 20%) (51, 72).

In connection with the problem of variable reflectances, it is pertinent to quote Gordon S. Kimball on the conclusion reached by the State of California:

"Some of California's paving is black top, and that which is concrete may eventually be blackened by oil stains. Coordinated studies on this problem brought the State to the conclusion that all pavement should be lighted as though it had the most adverse reflection value." (72)

Establishment of the pavement reflectance, the maximum hourly night-time total vehicle volume and the corresponding pedestrian volume at an intersection sets the minimum illumination levels, according to Table 10 previously cited. The lighting rating is then found by the relation

$$R_1 = 10 \left(\frac{\text{existing illumination level}}{\text{recommended minimum level}} - 0.5 \right)$$

Table 10

Current Recommended Average Horizontal Footcandles
(Lumens Per Square Foot)

Reflectance	Vehicular Traffic Classification*											
	Very Light (Under 150)			Light (150-500)			Medium (500-1200)			Heavy to Heaviest (1200 up)		
	Poor 3%	Normal 10%	High 20%	Poor 3%	Normal 10%	High 20%	Poor 3%	Normal 10%	High 20%	Poor 3%	Normal 10%	High 20%
Pedestrian Traffic												
Heavy (Main Bus. Sts.)	0.9	0.6	0.45	1.2	0.8	0.60	1.5	1.0	0.75	1.8	1.2	0.90
Medium (Intermediate)	0.6	0.4	0.30	0.9	0.6	0.45	1.2	0.8	0.60	1.5	1.0	0.75
Light (Residential)	0.3	0.2	0.15	0.6	0.4	0.30	0.9	0.6	0.45	1.2	0.8	0.60

*Numbers in parentheses are maximum vehicles per hour at night, total of all directions.

Based on:
American Standard for Street
and Highway Lighting
D 12.1 - 1953

where the range of R_1 is between zero and five. This rating gives no points for existing illumination which is equal to or less than half of the recommended minimum level in the particular situation.

The rating may be obtained graphically from Figure 10, 11 or 12, where the choice of figure depends on the pavement reflectance value. By selecting the appropriately numbered line to fit field conditions and entering the graph with the value which represents the average illumination level existing at the intersection, the rating is read directly.

Other Pertinent Characteristics

The two factors considered here complete the Physical Rating. Neither of these factors was considered on a par in importance with those previously discussed; as a result they were given lower maximum ratings and are briefly discussed.

Curbs

The existence or non-existence of curbs at an intersection as compared to the adjacent area and/or good design procedure is a cause for some concern. This rating is essentially a "consistency" measure. Is the design consistent with the surrounding area? With good design practice?

There appears to be no good objective method whereby this characteristic may be measured, so the subjective method below was perforce adopted. It will be noted that this

TABLE FOR SELECTING APPROPRIATE RATING LINE

PEDESTRIAN VOLUME	VEHICLE VOLUME	HEAVY (1200+)	MEDIUM (500- 1200)	LIGHT (150- 500)	VERY LIGHT (150-)
HEAVY (MAIN BUSINESS STS.)		(6)	(5)	(4)	(3)
MEDIUM (INTERMEDIATE AREAS)		(5)	(4)	(3)	(2)
LIGHT (RESIDENTIAL AREAS)		(4)	(3)	(2)	(1)

FIGURE 10. LIGHTING RATING FOR PAVEMENTS WITH LOW (3%) REFLECTANCE.

TABLE FOR SELECTING APPROPRIATE RATING LINE

PEDESTRIAN VOLUME	VEHICLE VOLUME	HEAVY (1200+)	MEDIUM (500- 1200)	LIGHT (150- 500)	VERY LIGHT (150-)
HEAVY (MAIN BUSINESS STS.)		(6)	(5)	(4)	(3)
MEDIUM (INTERMEDIATE AREAS)		(5)	(4)	(3)	(2)
LIGHT (RESIDENTIAL AREAS)		(4)	(3)	(2)	(1)

FIGURE II. LIGHTING RATING FOR PAVEMENTS WITH AVERAGE (10%) REFLECTANCE.

TABLE FOR SELECTING APPROPRIATE RATING LINE

PEDESTRIAN VOLUME	VEHICLE VOLUME	HEAVY (1200+)	MEDIUM (500- 1200)	LIGHT (150- 500)	VERY LIGHT (150-)
HEAVY (MAIN BUSINESS STS.)		(6)	(5)	(4)	(3)
MEDIUM (INTERMEDIATE AREAS)		(5)	(4)	(3)	(2)
LIGHT (RESIDENTIAL AREAS)		(4)	(3)	(2)	(1)

FIGURE 12. LIGHTING RATING FOR PAVEMENTS WITH HIGH (20%) REFLECTANCE.

score is on an "all or nothing" basis.

Table 11
Curb Rating

<u>Existing Condition</u>	<u>Points</u>
1. Intersection curbs exist where consistency and/or good design indicate a need for curbs.	3
2. Intersection curbs do not exist where there is considered to be no need for them from the standpoint of either good design or area consistency.	3
3. Intersection curbs do not exist where consistency and/or good design indicate a need for them.	0

Drainage

Occasionally the drainage characteristics of an intersection, or the lack of them, are of importance. The flooding of an intersection is a relatively rare occurrence in any event, so the maximum value of this factor has been set at two points. The rating is done in accordance with the following table:

Table 12
Drainage Rating

<u>Flooding History</u>	<u>Points</u>
1. There exists no record of the intersection having been flooded by a "ten-year storm."	2
2. The intersection floods occasionally.	1
3. The intersection floods chronically.	0

Factors Considered but Not Adopted

In addition to the factors described in the previous

paragraphs, several others were considered for inclusion, but were not adopted. These factors are listed and discussed below.

Pavement Width

This factor is used in the Indiana Sufficiency Rating Study, and, as part of an existing procedure, was considered for the urban intersection application (15). Capacity of a traffic lane in rural areas is highly dependent on the lane width (63), and so is the capacity of the urban lane proportional to its width (68). For the urban application, however, the calculated capacity was included in the Traffic Rating part of the intersection evaluation procedure, and separate and duplicate inclusion of this effect was not considered desirable.

Maintenance Economy

This factor was not included because of extreme complexity of evaluation of suitable data. Maintenance costs for long, reasonably-homogeneous rural road sections often are vigorously debated as to validity. Costs for spot locations such as intersections seem virtually hopeless. Furthermore this factor is highly correlated with pavement thickness and surface condition and these latter factors should indicate rather well the maintenance economy to be expected.

Estimated Life

Here again is a factor which is highly correlated with

thickness and surface conditions. High ratings on these latter factors should foretell a long estimated life; otherwise empirical survival curves must be used, and this is not a precise procedure by any means.

Traffic Adjustment

Adjustment of sufficiency rating factors by a traffic factor is a time-honored procedure. These adjustment formulae usually are of the form $A = B + \frac{B^2 - 100B (\text{Log } T - \text{Log } T_s)}{100}$ or something very similar, where A is the adjusted rating, B is the basic (unadjusted) rating, T is the existing traffic and T_s is the standard volume which is the comparison basis. The chief difficulty would be the evaluation of T_s for each intersection approach - a monumental undertaking. As in the case of the lane width, this factor enters through the Traffic Rating for an urban evaluation. It may also be recalled that where traffic volume was considered to be important in the Physical Rating, such volume was included directly. This was the case in rating the Lighting factor. It was thus concluded that where such an adjustment was really required it was already present; at all other places it was logical to ignore it.

THE TRAFFIC RATING

Where the Physical Rating of an intersection is concerned with visible, structural characteristics which affect traffic flow, the Traffic Rating is an attempt to measure "customer satisfaction" with intersection conditions as influenced by interference from traffic control devices, other vehicles and/or pedestrians. These influences are, in the main, variable over rather wide ranges during relatively short time periods, in contrast to the stability of physical factors.

Average Delay--The Rating Criterion

In establishing any evaluation system the first step is to select the factor that will best measure what is wanted. In this case, the question was what characteristic, or value, could be taken as the index of user satisfaction or dissatisfaction with the way he is served by a particular intersection. Some measure of vehicular delay seemed a logical choice in this connection. Where a driver passes through an intersection with little or no delay, he is pleased with the condition; if he is delayed more than a token amount his ire rises with the length of the delay until some point of frustration and resignation is reached. Beckman et al., state this briefly: "Conditions are good

if delay is small; they are bad if delay is large... we shall suppose that 'traffic conditions' are fully described by an assessment of the delays that occur" (7). Hall and George among many others have also written on travel time as a measure of congestion and level of service (28).

The principal types of vehicle delay may be classed as "stopped-time delay" and "average-travel-time delay." Other measures of delay might be obtained from the foregoing. D. S. Berry has said that, under certain conditions at least, stopped-time delay can be used as a basis for comparing total vehicular delay. Furthermore, stopped-time delay is relatively easily and precisely measured, whereas average-travel-time delay necessarily presupposes a known speed which represents the non-delayed condition (9).

Despite the apparent disadvantage to the use of the "average-travel-time delay" method cited above, in this case it proved advantageous, and was used.

The delay for any given vehicle was calculated as the difference between the time at which the vehicle was expected to arrive in the intersection if not interfered with by traffic control devices, other vehicles and/or pedestrians, and the actual time of entry after being subjected to any or all of the above influences. In equation form this was stated as:

Delay = Actual Entry Time - Expected Entry Time or,
briefly:

Delay = Time In - Time Due In.

A rating could have been based either on the total of such delays, or on the average delay. It was felt that the average delay is a better measure than total delay because it is a better measure of individual driver satisfaction. Furthermore, when the approach to a signalized intersection is at above-capacity volume, peculiar things happen to total delay relations that require close attention to assumptions and definitions to enable rational analysis of results. With the preceding considerations in mind, average delay per vehicle was adopted as the criterion for evaluating the Traffic Rating.

The Traffic Rating as developed is applicable to all intersections. The factors affecting delay at intersections, however, are many and variable depending upon type and amount of traffic control, volume, design, etc. It was not possible to evaluate all of these factors in this study. Furthermore, traffic delay studies have clearly indicated that major delays occur at signalized at-grade intersections. Consequently, this type of intersection was selected for detailed study relative to the Traffic Rating.

Fixed-Time Signalized Intersections

To evaluate average vehicular delay for an intersection approach with a given approach volume and known signal characteristics, certain assumptions were necessary. These are outlined below.

Assumptions Required

Uniform rate of arrival of vehicles was assumed. It has been found that free-flowing traffic follows the Poisson distribution, as reported by Greenshields et al. (26) and Matson, Smith and Hurd (52) among others. Nevertheless, at the volumes encountered at problem intersections, the average-headway concept seemed a logical and tolerable assumption, and has been used by Matson, Smith and Hurd in Traffic Engineering (52). The average headway is there designated "A," and is equal to $3600/V$ seconds, V being volume per hour. On the average, the first car is presumed to arrive midway in its time period, or $A/2$ seconds after the red signal phase begins. The same assumption was made for this study. Delayed vehicles were assumed to enter the signalized intersection from the approach at the time intervals reported by Greenshields, et al. The first car enters 3.8 seconds after the green begins, the second car enters 3.1 seconds after the first, and successive headways are 2.7, 2.4, 2.2, 2.1 2.1 seconds, the minimum value of 2.1 seconds being reached at the sixth delayed car.

The amber clearance interval was assigned an arbitrary length of three seconds, a typical value for urban installations. It was further assumed that if a vehicle were "due in" the intersection in the first two seconds of an amber light, it would enter, being too close to stop conveniently. If the vehicle were "due in" in the last second of the amber,

it was assumed to stop without entering. This appears to be slightly at variance with the findings of Greenshields where the actions of a limited number of "close-decision cars" are reported for such circumstances. The conclusion there was that "generally the amber signal causes no loss of time" (26). The situation as here assumed is slightly more stringent than Greenshields found to exist.

Delay was previously defined as time of actual entry less time of unimpeded entry. Such interval may only be evaluated when the vehicle actually enters the intersection, of course. For intersections with above-capacity approach volumes, the average delay was calculated for the first five-minute period that the overload existed, as more fully explained later. For those vehicles which were delayed but did not enter the intersection in the five-minute period, delay was determined from the time such vehicles were "due in" and the end of the five-minute period. The inclusion of such delays in the average caused the average delay " D_V " to be smaller for over-loaded volumes than it would have been if such vehicles were excluded.

Below-Capacity Operation

Total Delay for One Red Signal Phase. To obtain average vehicle delay where the approach volume is below intersection capacity, it was convenient to first find a procedure for determining total delay caused by one red phase of length "R." This total delay "T" is of course dependent on the

frequency of arrival of vehicles. Such a relation was found in Traffic Engineering (52) as:

$$T_n = nR - \frac{n^2 A}{2} + \Sigma \Sigma D_n \quad \text{where}$$

T_n = total delay, seconds, all n delayed vehicles;

n = number of vehicles delayed;

R = red phase length, seconds;

A = average vehicle arrival headway, seconds; and

D = departure headway, seconds.

It was convenient to tabulate D_n so that functions of D_n may be evaluated easily for any required number of cars; this is shown in Table 13. (The length of the table as given is due to the need for high values in column three for certain capacity calculations involving long cycle lengths and short red phases. These calculations are discussed later.)

Use of such a relation assumes a green interval sufficiently long to get rid of all those vehicles stopped as well as those arriving before the queue disperses. Use of the relation also requires knowledge of the value of " n ," the number of cars delayed. This may be figured for each case, or be obtained from Traffic Engineering also, where the relation is determined to be

$$n = \frac{R + 4.75}{A - 2.1}$$

and n , R and A are as previously defined.

The procedure was to first find the number of cars delayed, " n ," and then find the total delay incurred.

Table 13
Headways and Delays

Car Nr. "n"	Headway D_n	Delay to nth Car ΣD_n	Total Delay n Cars $\Sigma \Sigma D_n$
1	3.8	3.8	3.8
2	3.1	6.9	10.7
3	2.7	9.6	20.3
4	2.4	12.0	32.3
5	2.2	14.2	46.5
6	2.1	16.3	62.8
7	2.1	18.4	81.2
8	2.1	20.5	101.7
9	2.1	22.6	124.3
10	2.1	24.7	149.0
11	2.1	26.8	175.8
12	2.1	28.9	204.7
13	2.1	31.0	235.7
14	2.1	33.1	268.8
15	2.1	35.2	304.0
16	2.1	37.3	341.3
17	2.1	39.4	380.7
18	2.1	41.5	422.2
19	2.1	43.6	465.8
20	2.1	45.7	511.5
21	2.1	47.8	559.3
22	2.1	49.9	609.2
23	2.1	52.0	661.2
24	2.1	54.1	715.3
25	2.1	56.2	771.5
26	2.1	58.3	829.8
27	2.1	60.4	890.2
28	2.1	62.5	952.7
29	2.1	64.6	1017.3
30	2.1	66.7	1084.0
31	2.1	68.8	1152.8
32	2.1	70.9	1223.7

The following simple example illustrates the procedure.

Given $R = 35$ seconds and $V = 200$ vehicles per hour;

$$\text{then } A = 18.0 \text{ seconds. } n = \frac{35.0 + 4.75}{18.0 - 2.1} = \frac{39.75}{15.9} = 2.500$$

The total delay is given by the equation $T_n = nR - \frac{n^2 A}{2} + \sum D_n$.

$$\text{For } n = 2, T_2 = 2 \times 35 - \frac{2 \times 2 \times 18}{2} + 10.7$$

$$= 70 - 36 + 10.7$$

$$= 44.7$$

$$\text{For } n = 3, T_3 = 3 \times 35 - \frac{3 \times 3 \times 18}{2} + 20.3$$

$$= 105 - 81 + 20.3$$

$$= 44.3$$

It will be noted that the .500 fraction on "n" should be rounded down to get the critical vehicle since 44.7 seconds of delay for $n = 2$ is greater than 44.3 seconds for $n = 3$. In the case $R = 30$, $V = 300$, $A = 12.0$, n is determined as 3.51 cars. Using either 3 or 4 in the total delay equation results in $T_3 = T_4 = 56.3$ seconds. Fractional n values of .51 or greater were rounded up to the next integer.

Use of the preceding formulae are short cuts to the basic method shown below for the simple example $R = 30$, $V = 300$, $A = 12.0$ noted above:

<u>Car Nr.</u>	<u>Due In</u>	<u>Enters</u>	<u>Delay</u>	<u>Total Delay</u>
1	$6 = A/2$	33.8	27.8	27.8
2	$18 = 3A/2$	36.9	18.9	46.7
3	$30 = 5A/2$	39.6	9.6	56.3
4	$42 = 7A/2$	42.0	0.0	56.3

Through use of the methods just described, total delay due to one red signal phase was calculated for different red phase lengths and for different approach volumes. The results when graphed were as shown in Figure 13. It will be noted that no values are shown for volumes below 200 cars per lane per hour because it was felt that the uniform-arrival assumption becomes untenable at low volumes. Two hundred was arbitrarily selected as the cut-off point.

The graph lines were investigated for certain forms of relationship by plotting on semi-log and log-log papers, but in neither case did straight lines result.

Average Delay per Vehicle. In the preceding section, calculations were reported for delays due to one red phase, for various phase lengths and various approach volumes. This is the same as total delay per signal cycle (instead of red phase), if conditions are such that the backlog can be eliminated during the green phase. Conversion to average delay per vehicle in such case is simply a matter of multiplying total delay per cycle by the number of cycles per hour and dividing the result by the volume.

$$D_v \frac{\text{seconds}}{\text{vehicle}} = T \frac{\text{seconds}}{\text{cycle}} \times \frac{3600 \text{ seconds/hour}}{L \text{ seconds/cycle}} \times \frac{1}{V \text{ vehicles/hour}}$$

It will be recognized that $3600/V$ is equal to A , the average headway, and the expression may be simplified to $D_v = T \times A/L$.

For example, with $R = 30$, $V = 300$ and $A = 12.0$ as mentioned in an earlier example, T was determined to be 56.3

FIGURE 13. RELATION OF TOTAL CYCLE DELAY TO VOLUME FOR VARIOUS RED PHASE LENGTHS.

seconds. With a 60-second cycle, $D_v = 11.26 = 56.3 \times \frac{12}{60}$; for a 50-second cycle, $D_v = 13.51 = 56.3 \times \frac{12}{50}$. This procedure will hold until the green phase cannot discharge the delay caused by the red phase, at which time the approach is said to be operating above capacity.

Graphs showing the relation of average delay " D_v " to volume " V " are included in the Appendix to this report as Figures 42 through 50. These relations were calculated for cycle lengths of from 40 seconds to 80 seconds in five-second increments thus covering the common cycle lengths. Each graph has several lines which represent different red phase lengths. The red phases were chosen with five-second intervals also, over ranges considered appropriate to the cycle length.

The portions of the graphs reflecting below-capacity approach operation begin in the low-volume region and show relatively little increase in average delay with increasing volume, up to the points where the curves are discontinuous. These points represent capacity operation, and the location of these points is discussed in the following paragraphs.

Approach Capacity

Calculation of Capacity Volumes. It is possible to calculate, on the basis of the assumptions already made, at what volume an approach will reach capacity if the signal characteristics are known.

The first step in the procedure is to find the maximum number of vehicles that the specified green phase will permit

to enter the intersection. This may be done by subtracting non-entry time (red phase plus the final second of the amber clearance phase) from the cycle length. $E = L - (R + 1)$. From the third column of Table 13 the number of cars which will enter the intersection during such a period, on the average, may be determined. For long cycles and short reds, the number is sizable.

The average number of cars entering per cycle, just found above, is "n" in the relation $n = \frac{R + 4.75}{A - 2.1}$ previously utilized. By simple algebraic transformation $A = \frac{R + 4.75}{n} + 2.1$; n and R are now known and A may be readily found. Then: $3600/A$ equals V, the capacity volume in passenger cars per lane per hour.

For example, assume a sixty-second cycle and a forty-second red phase ($L = 60$, $R = 40$) on a certain intersection approach. Then the entry time available for the approach is nineteen seconds, made up of seventeen seconds of green plus the first two seconds of the amber clearance. Table 13 indicates that the 7th car will enter the intersection, on the average, 18.4 seconds after the green begins; the 8th car would enter 20.5 seconds after the green begins, but only nineteen seconds are available. So, on the average, the number of cars entering is 7 cars (using up 18.4 seconds) plus a fractional car determined by interpolating, i.e.: $\frac{19.0 - 18.4}{2.1} = 0.29$. On the average, 7.29 cars may enter in a nineteen-second entry period. Then: $A = \frac{R + 4.75}{n} + 2.1$;

$A = \frac{44.75}{7.29} + 2.1 = 8.24$ seconds and capacity volume $V_C = 3600/A = 3600/8.24 = 437$ passenger cars per lane per hour with the given signal characteristics.

In practice these capacity calculations should be made before the calculations for average delay per vehicle for the below-capacity approach volume condition, so that time will not be wasted making delay calculations for conditions which do not result in below-capacity operation.

Kind of Capacity Calculated. Before proceeding with an investigation of the agreement between capacities calculated by the method just outlined and those using conventional methods, it was necessary to determine what kind of capacity is represented by the results from this procedure.

It is believed that the method just outlined gives capacity values falling between the Possible and Practical capacities as customarily defined. This method is based on the assumption of uniform arrival at any volume. This is known to be merely an assumption; at some lesser volume due to unequally spaced arrivals, some vehicles will be delayed more than one cycle and the Practical Capacity definition will be fulfilled.

On the other hand, it is recognized that there is such a thing as the "pressurized intersection" (10, 77). Where congestion exists, drivers are more tense and alert and react more quickly so as to permit higher flow volume than under ordinary circumstances. In such cases the Possible

Capacity volumes will be higher than those calculated by this proposed method.

It thus appears that the calculated capacities are "Intermediate Capacity" values, directly analogous to values taken from the Highway Capacity Manual capacity curves (18, 68) and should be subject to approximately the same adjustment to get either Practical or Possible Capacity.

Comparison with Manual - Calculated Capacity Values.

The question may now be raised as to how well capacity values as calculated by the method just outlined agree with values calculated using the universally-accepted methods as outlined in the Highway Capacity Manual, the Manual of Traffic Engineering Studies and/or Circular 376 of the Highway Research Board's Highway Research Correlation Service (18, 57, 68).

In order to compare the methods it was first necessary to establish a common comparison basis. Investigation of Greenshields' work (26) indicated that his values for departure headways during a green signal phase are based upon the following conditions:

1. Passenger vehicles only - no truck or bus effect,
2. No turns effects - hence no pedestrian effect,
3. Sixty-foot approach street width,
4. Parking permitted on the approach,
5. Two-way traffic,
6. Intersection parking limitations unknown.

The capacities calculated by the proposed method outlined

were taken as appropriate for a standard ten-foot intersection approach lane.

Using the Highway Capacity Manual methods and the revised capacity graphs of Circular 376 (18, 68), capacity was calculated in the following manner. For "parking permitted, anywhere but downtown," with a thirty-foot approach width, approach capacity is given as 1750 vehicles per hour of green. The same volume could be carried on an eighteen-foot approach width in a fringe district with parking prohibited. The effect of permitting parking was thus taken as twelve feet of approach width for the intersection where Greenshields' values were established. The capacity for a twenty-two-foot approach (twelve feet for the parking effect plus ten feet for a standard intersection approach lane) was given by the curve as 1250 vehicles per hour of green. As discussed in the preceding section, this "curve" capacity value was considered directly comparable to the capacity as derived in this report, and consequently no adjustment was made to convert to Practical or Possible Capacity. However the Manual curve value had to be adjusted to the no-turn, no-commercial-vehicle basis inherent in Greenshields' data. This resulted in a twenty-five percent increase, ten percent being added for elimination of commercial vehicles, a like amount for left-turn elimination, and five percent for right-turn elimination. The Manual capacity for the single ten-foot approach lane, parking permitted, fringe area, was thus

increased to 1562.5 vehicles per green hour, or, say, 1560 vehicles per green hour.

To compare methods, a fifty-second cycle was selected, with red lengths of twenty, twenty-five and thirty seconds. The Highway Capacity Manual cautions against using amber time as part of the entry time, hence the corresponding entry times were twenty-seven, twenty-two and seventeen seconds. Then the comparison is as shown in Table 14.

Table 14
Comparison of Calculated Capacities
for 50 Second Cycle Length

Red Phase	Manual Method			Proposed Method	
	Entry Time	Time Factor	Calc. Capacity	Entry Time	Capacity
20	27	27/50	844	29	867
25	22	22/50	688	24	695
30	17	17/50	531	19	524

Thus, when the calculated capacity of 1560 cars per green hour was converted to cars per clock hour by multiplying by the ratio of green phase length to total cycle length, the resulting capacities agreed closely with those computed earlier.

It will also be noticed that the proposed method reflects the added efficiency of long green phases, and vice versa. Indicated capacities are higher than those obtained by Manual methods where greens are long, and lower where

green phases are short.

This comparison indicates that the "intermediate," or graph, capacity for a single standard ten-foot lane of passenger cars, inhibited only by each other and signal characteristics, is approximately 1560 cars per hour. This, it will be noted, is considerably in excess of the Basic Capacity of 1500 passenger cars per twelve-foot lane, or 1250 cars per standard ten-foot lane currently stipulated by the Highway Capacity Manual. Recent writings indicate, however, that the above Manual figures will be revised in the forthcoming new edition. With the curves of Circular 376 replacing those of the original Manual (18, 68) associated data must necessarily also be revised. Webb and Moscowitz report shorter headway times and higher capacities in California than the Manual would indicate (77). Also George Sagi reports a theoretical value of 1714 vehicles (cars) per lane per hour for the situation where "conditions are ideal and when the steady flow of vehicles is periodically interrupted by stops and where the go time is also limited" (66). Sagi's value of 1714, if divided by 110% to reduce it to "intermediate capacity," gives a value of 1558 vehicles (cars) per hour of green. Oscar Sutermeister found intersection approaches operating at up to 100 percent above Practical Capacity as presently computed (75). Nor are such findings restricted to this country. Various papers from foreign lands indicate similar results, although in evaluating such

results cognizance has to be taken of differing conditions such as smaller vehicles, leading amber lights, restriction of certain maneuvers, etc. (6, 43).

The "Standard Lane" Concept. For better or worse, the capacity volume of 1560 cars per hour of green per ten-foot approach lane considers only two types of interference, namely signal delay and passenger car mutual interaction. Unfortunately, such situations are not common, and steps must be taken to face up to the facts of urban traffic life.

The most satisfactory method presently available for evaluating the various disruptive influences found at the typical intersection is the Manual approach-capacity calculation, based on existing conditions rather than the highly-theoretical specialized case just discussed. Such capacity results are in terms of vehicles (not cars) per hour of green and include the effects of all hindrances to flow that the present state of the art permits. It may be noted in passing that a new Highway Capacity Manual is in preparation, with the effects of forty-five variables on intersection capacity being investigated by analyses done on electronic computers. O. K. Normann has said ".... it is expected that far more will be known regarding reasons for variation in intersection capacities than has been known in the past. It is expected that it will be possible to establish reliable factors for many of the forty-five different variables. The effect of the more important ones now covered in the present manual

- turning movements, commercial vehicles and bus stops - and many others not now included will be accurately determined from the large volume of data available." (59)

If the calculated intersection approach capacity is divided by 1560, the standard lane capacity, a value is obtained which may be called the "effective number of lanes" of traffic which may flow. If, in turn, the actual volume of flow is divided by the "effective number of lanes" the result is that part of the total flow which may be considered to flow in one standard ten-foot approach lane.

$$\text{Std. Lane Volume (cars/hr)} = \frac{\text{Total Approach Volume (vehicles/hr)}}{\frac{\text{Calc. Capacity (veh/green hr)}}{1560} \text{ (cars/green hr)}}$$

Naturally, the characteristics of the total traffic volume are those used to calculate the approach capacity.

The equation may be written in different form as

$$\frac{\text{Standard Lane Volume}}{1560} = \frac{\text{Total Volume}}{\text{Calculated Capacity}}$$

In the latter form it is readily seen that finding the standard lane volume is a ratio process. Both sides of the equation are volume/capacity ratios, one representing an existing condition, the other a theoretical condition. The standard lane volume, which is in cars per hour, is the same multiple of 1560 as the total volume is of the calculated capacity.

It was necessary to establish a common measure of volume such as the standard lane volume (in terms of passenger cars

per hour) to provide a common basis for volume - delay relations because the departure data utilized are valid only for the case where the traffic is made up entirely of passenger cars. Note that the "standard lane" concept (cars only) was used in the below-capacity delay calculations.

Determination of the standard lane volume may be expedited by graphical means such as the N-chart in Figure 14.

Above-Capacity Operation

With the selection of average delay per vehicle as the factor to be evaluated as a measure of user satisfaction, one of the big problems connected with above-capacity operations was solved by definition. The fact is that there are several types or measures of delay which might have been used but none was simple, and none except average delay gave a graphical relation of the shape considered desirable.

Adoption of the Five-Minute Delay Period. Above-capacity operation is by definition that case where more vehicles arrive during the cycle than may enter the intersection. By the nature of such operation, as long as the volume remains constant, the backlog increases with every signal cycle. There is no single value for average delay as in below-capacity operation; whatever value may be calculated for average delay depends, among other things, on how long the overload has been in existence. The average delay increases as the overload gets larger.

Clearly, if a measure of average delay was to be estab-

FIGURE 14. DETERMINATION OF STANDARD LANE VOLUME.

lished, it had to be applied on an arbitrary basis. For this purpose a five-minute period was selected, and average delay was calculated for the first five minutes of overload. For those vehicles which arrived, were delayed, and entered the intersection within the five-minute period, the delay was calculated the same as in below-capacity operation, i.e.: delay equals time in minus time due in.

If the vehicle had not entered the intersection at the end of the five-minute interval, its delay was calculated as 300 seconds minus time due in the intersection. The delay for these vehicles was hence understated, and caused the average delay to be somewhat lower than it would have been if such vehicles were ignored altogether.

Electronic Computer Utilized. Earlier, for all reasonable combinations of cycle lengths and red phase lengths, the capacities were calculated. In these calculations, the time-spacings "A" of the capacity flows were also calculated as a necessary step. Having the same combinations of cycle lengths and red phases, and knowing what headway spacing "A" would set up above-capacity operation in each case, it was not difficult to program the calculation of average delay for electronic computation. The average delays for above-capacity operation shown later in this report were computed on Purdue University's Datatron.

Some such method was a virtual necessity because numerous points were required to fix each curve accurately, and there

were seventy-two curves. In addition, as the volumes built up, up to two hundred cars needed to be considered in the arbitrary five-minute period to compute the average delay for one volume and one signal condition. This all added to much tedious labor if advantage had not been taken of modern methods.

Figure 15 shows the flow diagram utilized in the computation. The diagram shows that, once started with an L, R, A data triplet, the routine goes until the average delay is calculated. Then the program calls for a reduction in A (increase in volume) and starts over on another calculation using the same L and R values. Reductions in A depend on the size of A - a change of 0.2 second in A when A is 18.0 seconds amounts to two vehicles per hour. A change of 0.2 second in A from 2.2 to 2.0 seconds represents approximately one hundred sixty vehicles per hour. Amounts of the changes in A and division points where they occurred were calculated to give volume changes between delay calculations of from twenty to forty vehicles per hour as closely as feasible. When the average delay exceeded a pre-established maximum at the end of the five-minute period, the computer abandoned that L, R and reduced A set and picked up the next data triplet. If there was no triplet left, the machine stopped.

The data calculated for above-capacity approach operation for the various cycle lengths and red phase lengths previously referred to are also shown on the graphs in the Appendix, Figures 42 through 50. In the section on below-

FIGURE 15. FLOW DIAGRAM FOR ELECTRONIC COMPUTATION OF AVERAGE DELAY AT ABOVE-CAPACITY APPROACH VOLUMES.

capacity operation it was mentioned that such operation was represented by the flat portions of the curves, up to the points of discontinuity which represent capacity operation. The points representing above-capacity operation define the curves at the higher volumes and produce the steeper curve portions, where average delays increase rapidly with relatively small volume increases. These above-capacity curves were terminated when the average delays reached sixty seconds per vehicle in accordance with the rating scale adopted as described in the next section.

Establishing the Rating Scale

A rating scale should be suitable, convenient and logical. With these criteria in mind a scale was sought to fit the various graphs. It should be the same scale on all graphs; if a user sustains a long delay, his feelings aren't dependent on the control characteristics. A graph of average delay at a signalized intersection was examined for a fifty second cycle with a twenty-five second red phase, this appearing to be a representative, average condition (see Figure 16). When this approach was flowing at half-capacity volume the average delay was found to be 10.3 seconds. This value was rounded to 10.0 seconds and adopted as the origin of the rating scale. Any intersection condition resulting in an average delay of 10.0 seconds or less was rated 100 points.

At capacity, the beforementioned curve showed delay of

FIGURE 16. ESTABLISHING THE RATING SCALE.

14.5 seconds. If the rating scale were fifty seconds long, running from 10 to 60 seconds, this resulted in approximately ten percent (nine, in this case, to be precise) of the rating being responsive to below-capacity delay increases.

These characteristics were deemed appropriate and the rating scale was adopted as described. Average delays of 10.0 seconds or less rated 100 points. Average delays in excess of 10.0 seconds decreased the rating by two points for each second of such excess. If the delay reached 60.0 seconds or beyond, the excess over 10.0 was 50.0 and the rating was zero. No negative ratings were provided.

The delay approach-rating relationships just discussed are considered applicable to the Traffic Rating of all intersections regardless of the type of control or other cause of delay. The delay was evaluated in this study, however, only for certain signalized intersection approaches and the expected delays at these types of intersections are discussed and shown in the following sections of this report. The evaluation of the expected delays at other types of intersections can be done in a similar manner. The relationship between traffic volume and average delay for intersections which are not controlled by fixed-time signals will not be the same as that shown in Figure 16 but may be expected to be similar.

Rating Fixed-Time Signalized Intersections

Establishment of Transition Curves

Examination of the typical delay-volume curve for intersections with fixed-time signals shown in Figure 16 indicates that average delay increases at a rather slow and reasonably uniform rate until capacity is reached. The curve is discontinuous at the point representing the capacity condition, the first few cars of the over-capacity volume adding large delays and affecting the average delay greatly.

While such condition may accurately reflect delays at this type intersection it is not a desirable characteristic for a rating curve. It was considered that the Traffic Rating should reflect a measure of remaining trouble-free operation. As the standard lane volume approaches the capacity value, it was believed that the rating should begin to be reduced to reflect the fact that the satisfactory operating life of the intersection is limited, and before too long remedial action will be needed.

A proper theoretical approach to this problem would seem to be to establish some percentage of capacity at which reduction of rating would begin, making the transition curve tangent to the theoretical curve at this point. At some other percentage of capacity (greater than 100%), the transition would again join the theoretical curve, and the discontinuous part of the theoretical curve would be smoothly and reasonably bridged.

The principal drawback to such an approach can only be suspected from Figure 16, but becomes apparent on plotting the curves for the various red phase lengths evaluated at any given cycle length (see the Appendix). Where red phases are short compared to cycle length, capacities are relatively high; the first overloading vehicles increase delay but it is spread over a larger number of vehicles and the effect is not drastic. As red lengths increase, capacity drops. When capacity gets as low as, say, two to four hundred vehicles per hour, the same few overloading vehicles have a tremendous effect on the average delay, which increases rapidly with only a slight volume increase.

After some investigation, it appeared that a transition beginning at seventy percent of capacity and ending at one hundred ten percent of capacity would be feasible. Further criteria were needed, however, for with only the end points specified, many curves could be fitted to a single condition, producing highly variable results. It was decided to make the transition a circular curve combined with a tangent. The circular curve begins at seven-tenths of capacity and is of such radius that it ends at the tangent which is drawn to the theoretical curve at one and one-tenth times capacity.

This method of providing transitions resulted in steep curves at low capacities and more gentle transitions at higher capacities. This was in keeping with the severity of results of traffic increase, and seemed logical.

The Rating Curves and Their Use

The rating curves are shown in Figures 17 through 25, for cycle lengths of from forty to eighty seconds in five-second increments. On each figure are shown lines representing red phase lengths deemed appropriate, also in five-second increments. Interpolation is required where the red phase is not a multiple of five seconds.

To use the rating scale, the signal characteristics, either existing or proposed, and the standard lane volume are required.

There remains the problem of establishing the standard lane volume with which to enter the chart, however. What volume is it that is to be multiplied by 1560 and the product divided by the calculated approach capacity? It is obvious that any one of several design hour volumes might be justifiable, but the actual volume adopted for conversion to standard lane volume was that hourly volume calculated from the highest 15-minute volume measured on a typical day on the approach being rated.

Measured Delay at Fixed-Time Signalized Intersections

Verification of Theoretical Relations

"Logic is an organized way of going wrong with confidence" is a quotation attributed to Charles Kettering (45). To guard against such occurrence in this case, it was considered necessary to investigate a number of existing signalized inter-

FIGURE 17. INTERSECTION APPROACH TRAFFIC RATING, 40 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 18. INTERSECTION APPROACH TRAFFIC RATING, 45 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 19. INTERSECTION APPROACH TRAFFIC RATING, 50 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 20. INTERSECTION APPROACH TRAFFIC RATING, 55 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 21. INTERSECTION APPROACH TRAFFIC RATING, 60 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 22. INTERSECTION APPROACH TRAFFIC RATING, 65 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 23. INTERSECTION APPROACH TRAFFIC RATING, 70 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 24. INTERSECTION APPROACH TRAFFIC RATING, 75 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

FIGURE 25. INTERSECTION APPROACH TRAFFIC RATING, 80 SECOND CYCLE AND VARIOUS RED PHASE LENGTHS.

section approaches to determine if, in fact, actual traffic behavior compared with the theoretical volume-delay relations derived.

Choice of Intersections to Be Studied

The first step in the verification attempt was a study of local intersections to determine which had characteristics such that necessary data could be obtained over appropriate ranges without the introduction of extraneous analysis factors.

Since delay was previously defined as time of entry less time due in the intersection, it was first of all necessary to have an intersection approach sufficiently long and uncomplicated that such delay might be accurately measured. It was not found possible to fulfill this requirement to the desirable extent in every case because zones long enough to be unaffected by most traffic back-ups usually included a side street with associated problems.

The second major requirement was that the approach flow at relatively high volume, at least during some portions of the test period. Approaches that overloaded for short periods were desired so that data might be gathered for the above-capacity portions of the respective curves.

In other respects, it was desired to keep the approaches as simple and uncorrupted as possible.

Locations chosen, with appropriate features noted, are shown in Figures 26 through 29.

At Fourth and Owen, signal characteristics were such

SIGNAL CHARACTERISTICS			60 SEC. CYCLE	65 SEC. CYCLE
57 %	APPROACH	GREEN	34.2 SEC. G.	37.1 SEC. G.
6 %	"	AMBER	3.6 SEC. A.	3.9 SEC. A.
37 %	"	RED	22.2 SEC. R.	24.0 SEC. R.

CALCULATED CAPACITY = 1097 = 1100 VEH./GREEN HR

BASED ON 4 % TRUCKS

8 % LEFT TURNS

7 % RIGHT TURNS

FIGURE 26. OBSERVED APPROACH CONDITIONS, FOURTH ST. AT OWEN ST., LAFAYETTE, INDIANA.

SIGNAL	CHARACTERISTICS	60 SEC CYCLE	70 SEC CYCLE
30 %	APPROACH GREEN	18 SEC G	21.0 SEC G
5 %	AMBER	3 SEC A	3.5 SEC A
65 %	RED	39 SEC R	45.5 SEC R

CALCULATED CAPACITY = 1248 = 1250 VEH / GREEN HR

BASED ON 3 % TRUCKS

0 % LEFT TURNS

11 % RIGHT TURNS

FIGURE 27. OBSERVED APPROACH CONDITIONS,
NORTHWESTERN AVE. AT STATE ST.,
W. LAFAYETTE, INDIANA.

SIGNAL CHARACTERISTICS			60 SEC. CYCLE	80 SEC. CYCLE
44 %	APPROACH	GREEN →	26.4 SEC. →	35.2 SEC. →
25 %	"	GREEN	15. SEC. G.	20. SEC. G.
4 %	"	AMBER	2.4 SEC. A	3.2 SEC. A
71 %	"	RED	42.6 SEC. R	56.8 SEC. R.

CALCULATED		CAPACITY	=	1915	VEH./GREEN	HOUR
BASED	ON	4 %	TRUCKS			
		1 %	LEFT	TURN		
		0 %	RIGHT	TURN		
AND	ACTUAL	RIGHT	TURNING	VOLUMES.		

FIGURE 28. OBSERVED APPROACH CONDITIONS, SOUTH ST.
AT MAIN ST., LAFAYETTE, INDIANA.

FIGURE 29. OBSERVED APPROACH CONDITIONS, MAIN ST. AT SIXTH ST., LAFAYETTE, INDIANA.

that, regardless of high flow volumes, capacity was reached very rarely, and for very short periods. The main extraneous disruption to traffic noted there was that traffic turning left (south bound to east bound) into Central occasionally held up those not wishing to make such a turn. Also, in the data analysis stage, it became obvious that the signal operated on a 60-second cycle one day and 65-seconds the remainder of the time observed. This was taken into account and did not appear to influence experimental results additionally.

At State and Northwestern there was also an intervening side street in the measurement zone, which appeared to cause no difficulty. The signal was a three-phase, two-dial operation, and left turns were prohibited on the approach investigated. Note the long no-parking zone on the approach and the added lane on the opposing exit.

At South and Main, the signal was four-phase, two-dial, providing a leading right-turn arrow. Lane markings were provided. This intersection was studied to see if method verification could be had for such a separate-turning-lane condition since the present Highway Capacity Manual is not entirely clear on how the capacity of such an intersection should be evaluated. Capacity was reached at relatively low volumes here because of the four phase operation.

At Main and Sixth, the approach studied was one-way. It was considered desirable to see if the standard lane

capacity value of 1560 was applicable in such case. This approach was the only one of the four with no intervening side street. It was found desirable to position the field parties and radar speed meter on the left side of the street to prevent masking the faster left-hand lane traffic by slower-moving trucks and buses which customarily kept right.

All measurement zones were approximately level, and although two were at the top of long steep hills, the beginning of the zones were sufficiently far from the hill crests that no difficulty was experienced from delayed heavy trucks.

Care was also taken to choose approaches where the vehicles could be considered to arrive uniformly, without regimentation from preceding signals.

Procedure

With average delay on the "time in" less "time due" basis, it was necessary to evaluate delay in the field on the same basis. To accomplish this, time of passage was observed from a stop watch at the advance station (Station A) for each vehicle passing. The speed was also observed for each vehicle at "A," using an Electromatic radar speed meter. Knowing the length of the observation zone, the "time due" was evaluated as the time of passage at "A" plus a function of distance divided by speed. Actual time of entry at the base station (Station B) was recorded from a second stop watch, both watches having been started simul-

taneously so that they read the same at any instant of time.

The final relation for delay for a single car was

$$\text{delay} = \text{time B} - \left(\text{time A} + \frac{D \text{ (feet)}}{R \text{ (mph)}} \times \frac{60 \text{ mph}}{88 \text{ fps}} \right)$$

or

$$\text{delay} = \text{time B} - (\text{time A} + .68D/R).$$

For any zone D was a constant, and time-of-transit tables were prepared for all reasonable speeds.

Individual delays were then grouped and averaged as desired.

Field Organization

Field work was begun with a crew of six, but it shortly became very apparent that more men were needed. To get full coverage of data it was finally necessary to use eight persons, with the organization and duties as outlined below.

Four men comprised the party at Station A. The first man was the timer, who called out the second ("ten, twelve, fifteen") that each vehicle passed Station A. The radar man observed the speed from the meter and reported it immediately following the associated time of passage. At lower flows, the radar operator also glanced up and read the last two characters (either numbers or letters) of the vehicle license plate, and reported these with the speed as a four character group, such as "twenty-three - eighty-five," twenty-three being the speed and eighty-five being license characters for identification, so that appropriate data could be matched between Stations A and B. The third man, a very busy one,

was the recorder whose duty it was to get down on a record form all the spoken times, speeds and identifications. At higher volumes this proved too much to expect - the recorder simply could not go fast enough. So, he was relieved of the license-plate recording and restricted to times and speeds with only an occasional license plate for tie-in purposes. License-plate recording was assigned to the fourth man whose sole job it was. These extra field notes were kept on a tablet and entered on the regular record sheet during lulls or at the end of the day. The importance of an occasional license recording by the regular recorder cannot be over-emphasized. Otherwise if either man missed a car, licenses would be matched with wrong times and speeds, and there would be no possible way of locating or minimizing such an error.

At Station B there were a timer, a principal recorder and an auxiliary recorder with duties the same as noted for Station A. With no radar speed to record it might be thought that one recorder could have done the job, but it must be remembered that entry headways are in general shorter than arrival headways. That is, the work was concentrated into short periods, and, again, at high volumes, one recorder couldn't keep up.

The eighth man, located at Station B, had duties distinctly separate from the others. He conducted a classification count by five-minute periods, recording the numbers of passenger

cars and "other vehicles" that proceeded straight through the intersection, turned right or turned left. The "other vehicles" classification included buses and trucks that had characteristics different from cars. For this purpose, any truck with single rear axle and single tire on each side was classed as a car; any multiple-rear-axle truck or truck mounting dual tires in the rear was classed with "other (than passenger) vehicles." Such data were necessary for determining the calculated approach capacities under existing traffic conditions, and whether such conditions varied appreciably from time to time during the periods each intersection approach was under observation.

Particular attention was devoted to impressing upon both timers and both recorders, and especially those at Station A, the necessity for keeping accurate and complete record of the minutes on the data sheet. If this was negligently done much valuable information was rendered questionable and/or useless because of the difficulty or downright impossibility of accurately interpreting such incomplete data.

Where traffic entered the intersection in more than one lane, or passed the A Station in double lane, as at Main and Sixth Streets, emphasis was placed on taking data in chronological order of passing the reference point, without regard for the lane the vehicle happened to occupy.

Field equipment included the Electromatic radar speed

meter, two stop watches, four clip boards for the recorders, one tally board with multiple counters for classification, recording forms and pencils, voltmeter, extra radar batteries and assorted chairs, stools and tables.

It was convenient to set the radar up in the street, in front of a parked car to hide it, and read the speed of the vehicle as it was going away from the meter. It is felt that most drivers never saw the radar. However the data crews were not so readily hidden and caused some curiosity among passers-by.

Analysis of the Data

Twelve afternoons and fractions of two others were spent in the field gathering data. From all of this effort 479 five-minute periods of technically-perfect data were obtained, which figure does not include those periods when multi-dial controllers were changing dials, resulting in re-synchronization with the consequent one or two odd-length cycles. The 479 value also does not include those five-minute periods where satisfactory delay data were obtained on less than fifty percent of the vehicles passing A Station, "satisfactory delay data" being more fully explained below.

During the 479 five-minute periods, 16,472 vehicles passed the A Stations, and 13,437 of these were satisfactorily matched with B Station data. The 13,437 vehicles on which delay calculations were made accumulated a total of 318,751 seconds (about 88-1/2 hours) of delay according to calcula-

tions which were checked mathematically at every step and reviewed for reasonableness at every step. No claim is made for 100% accuracy, but it is felt that every precaution was taken and the data truly reflect actual field conditions.

Early in the calculations, which followed the form set forth earlier in the section on Procedure, it became apparent that certain restrictions needed to be imposed on the calculation of delay for the individual vehicle. In certain cases, negative delays were calculated. It was concluded that such delays of small magnitude were tolerable and were attributable to slight errors in reading the radar speed and/or time of passage at either station. At low speeds in particular, a one mile-per-hour error affected the time of transit by two or three seconds. The tolerable limit for negative delays was set at minus three seconds. Such values were accepted as resulting from random error, and were figured to average out in the long run. A value below minus three seconds resulted in rejection of that vehicle for error of undeterminable cause.

Negative values around sixty seconds suggested an arithmetical error, or, if persisting over a group of several cars, that one recorder erred by one minute in recording time. The analyst in such cases had to be a good detective.

To eliminate another error source, it was necessary to insure that the radar speed as measured represented the driver's uninhibited and freely-selected speed. In some

instances the intersection backlog, or something in the approach zone, caused drivers to slow before passing the radar at A Station. After some consideration, eleven miles per hour was selected as the minimum value that would qualify as uninhibited approach speed. Any radar speed of ten or fewer miles per hour resulted in that vehicle's being eliminated from the delay calculations. Use of such unrealistic low speeds would have resulted in estimated times of arrival that were later than they should have been. Such drivers speed up again the first chance they get, and delays would be understated because parts of such delays are charged to transit time. Most drivers actually traverse the zone faster than estimated in such cases.

Occasionally it was obvious from the delay value for an individual vehicle that the driver had done something other than proceed through the approach in the normal manner, such as park for a short period to run an errand. Where such values could be detected, the vehicles were eliminated.

Vehicles not eliminated by the above rules, and their delays, were summed in five-minute periods, and the average delay per vehicle was calculated by dividing the total delay by the number of cars contributing to the total. If this number of cars for which delay values were considered valid was not at least fifty percent of the total of all vehicles passing A Station in that period, the entire period was dropped from consideration. Several such periods were not

included in the 479 periods previously stated to be valid.

The total number of vehicles passing A Station (whether matched at B Station or not) was considered the approach volume, and this figure was multiplied by twelve to put the volume on an hourly basis. The hourly volume rate was then converted to standard lane volume by multiplying by the ratio 1560/calculated approach capacity.

It is worthy of note here that the Northwestern Avenue approach, by virtue of its long no-parking zone and the added exit lane, qualified for capacity calculation as a street with parking prohibited, and subject to two-lane operation. Field observation of traffic behavior, however, showed that drivers did not use it in this fashion. Only when delays were high and the pressure was really on were both lanes utilized fully.

Figures 30 through 36 show the relation of the field delay data to appropriate theoretical curves. Figures 32A and 33A show these relationships where the standard lane volumes were figured considering Northwestern as a two-lane approach with no parking. Agreement of field delay values with theoretically-predicted values was very poor for these cases. When capacity was calculated in accordance with the manner in which the approach was actually used, experimental results were grouped around the theoretical curves as shown in Figures 32B and 33B.

Although the overall agreement of the data points with

FIGURE 30. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, FOURTH AT OWEN, 60 SECOND CYCLE, 22 SECOND RED PHASE.

FIGURE 31. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, FOURTH AT OWEN, 65 SECOND CYCLE, 24 SECOND RED PHASE.

FIGURE 32. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, NORTHWESTERN AT STATE, 60 SECOND CYCLE, 39 SECOND RED PHASE. TWO CASES.

FIGURE 33. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, NORTHWESTERN AT STATE, 70 SECOND CYCLE, 45 SECOND RED PHASE. TWO CASES.

FIGURE 34. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, SOUTH AT MAIN, 60 SECOND CYCLE, 43 SECOND RED PHASE.

FIGURE 35. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, SOUTH AT MAIN, 80 SECOND CYCLE, 57 SECOND RED PHASE.

FIGURE 36. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, MAIN AT SIXTH, 65 SECOND CYCLE, 35 SECOND RED PHASE.

the various curves was considered surprisingly good in general, deviations of the observed points from the curves were, in cases, quite large. It was thought that the deviations might be reduced and agreement maintained, or perhaps improved, if average delays were calculated over ten-minute periods rather than five-minute periods. This analysis was made. It resulted in a slight loss of usable data because only 232 ten-minute groups were considered valid for analysis purposes. Five of these groups contained one five-minute period which was rejected in the five-minute analysis because of having less than 50% of the vehicles satisfactorily matched, yet when such periods were combined with their companion periods, the 50 percent restriction was met for the overall sample. Since this was the case, and these data would have been included if the ten-minute analysis had been made first, they were included here. Twenty five-minute periods were dropped because they could not be combined into full ten-minute periods.

The ten-minute analysis covered 16,107 vehicles, of which 13,070 were satisfactorily matched, and the total delay was figured to be 307,360 seconds (85.4 hours).

The results of this analysis are shown in Figures 37 through 41. It seemed apparent by inspection that variability was decreased by the ten-minute data grouping, but a quantitative basis for this conclusion was considered desirable. Since it was the average deviations of the data points from

FIGURE 37. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH
TEN-MINUTE DATA GROUPING, FOURTH AT OWEN, 60
SECOND CYCLE, 22 SECOND RED PHASE.

FIGURE 38. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH TEN-MINUTE DATA GROUPING, FOURTH AT OWEN, 65 SECOND CYCLE, 24 SECOND RED PHASE.

FIGURE 39. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH TEN-MINUTE DATA GROUPING, NORTHWESTERN AT STATE, 60 AND 70 SECOND CYCLES, 39 AND 45 SECOND RED PHASES.

FIGURE 40. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH TEN-MINUTE DATA GROUPING, SOUTH AT MAIN, 60 AND 80 SECOND CYCLES, 43 AND 57 SECOND RED PHASES.

FIGURE 41. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH TEN-MINUTE DATA GROUPING, MAIN AT SIXTH, 65 SECOND CYCLE, 35 SECOND RED PHASE.

the corresponding theoretical curves that was of interest, the following relation was adopted as a measure of the variability:

$$V_t = \frac{\sum (D_v - D_t)^2}{n}$$

where V_t is the variance of the points around the theoretical curve,

D_v is the average vehicle delay over the time period being evaluated,

D_t is the theoretical delay as given by the appropriate curve,

$(D_v - D_t)$ is the deviation of the actual value from the expected value, and

n is the number of data points being examined.

The above-indicated variances were computed for all observed approaches for every signal cycle length for both five-minute and ten-minute data groupings. This resulted in eighteen variance comparisons. In addition, where similar signal conditions existed at an intersection over several days of observation, pooled variance estimates were calculated, resulting in six more variance comparisons. In seventeen of the eighteen daily variance comparisons, the ten-minute variance was smaller than the five-minute variance. For the pooled estimates, all six ten-minute variances were smaller than their corresponding five-minute variances.

Thus in general the variations of the ten-minute means around the theoretical curves were found to be less than the

corresponding variations of the five-minute means.

Conclusions of the Verification Study

As a result of the field verification study, the following conclusions were drawn.

First, it was concluded that with the appropriate standard lane volume, the theoretical curves give acceptable estimates of the average vehicular delays to be expected under stipulated signal conditions.

Second, care must be taken to calculate approach capacity on a basis which conforms to the way the intersection is actually used. Northwestern Avenue at State Street was such a case.

Third, more precise methods of calculating capacity would be most valuable in the application of this method, particularly at intersections with rather unusual characteristics such as separate turning lanes and/or separate turning indications. South Street at Main illustrated this. It is expected that the imminent new edition of the Highway Capacity Manual will be most helpful in this connection.

Fourth, on the basis of the previously-discussed calculations, it was concluded that data grouped in ten-minute periods had less variance about the respective theoretical curves than did the data from five-minute periods. The ten-minute data thus "fit" the curves better.

THE INTERSECTION RATING

The basic unit for rating purposes was determined to be the intersection, inasmuch as all elements of the Physical Rating were based on the intersection as a unit, as opposed to separate ratings for the various approaches. It was thus required that the Traffic Rating be transformed from an individual-approach basis to some sort of composite rating that could be taken as representative of the characteristics of the intersection as a whole.

A method for conversion of Traffic Ratings for the individual approaches to an overall composite intersection Traffic Rating was established and is discussed in this chapter. The Physical and composite Traffic Ratings were then related to determine the Intersection Rating and a table for easy solution of this relationship is provided.

In proceeding in this fashion, one danger should be recognized so that it may either be provided for, or else be ignored with full recognition of the possible consequences. This is the danger that any composite rating may mask an extremely low rating in any of the components - a rating which is so low as to indicate that an emergency exists in the specific area and immediate action is therefore warranted, tolerable combined rating notwithstanding. In Highway Sufficiency Ratings currently in use, this problem is recognized

and handled by the establishment of cut-off values for critical elements, so that a lesser rating puts the section on a "critical list" regardless of the over-all rating (1, 12). Separate recording of constituent ratings and use of special lists should solve this problem.

Conversion of Approach Traffic Ratings to Intersection Basis

To convert the various approach Traffic Ratings to a corresponding intersection Traffic Rating, the most logical method appeared to be that which weighted the approach ratings in accordance with their respective volumes of traffic. This method was adopted, and the following relation was used:

$$TR = \frac{V_1 R_1 + V_2 R_2 + \dots + V_n R_n}{\sum_{i=1}^n V_i}$$

where TR is the intersection Traffic Rating,

V_i is the volume on the "i"th approach,

R_i is the approach Traffic Rating of the "i"th approach,

n is the number of intersection approaches, and

$\sum V_i$ is the total volume through the intersection.

This conversion method is general, and may be used irrespective of the type of traffic control existing at an intersection.

It may also be pointed out that where an anticipated or proposed change at an intersection involves only traffic control devices, such as change from stop signs to signal

control, or change in characteristics of existing signal control, the composite Traffic Rating may be used by itself to indicate the relative merits of various proposals. The Physical Rating, remaining unchanged, need not be considered at this point; however, as is set forth in later sections, a change in the intersection Traffic Rating resulting from control changes or any other cause does not change the Intersection Rating by a like amount.

Intersection Rating Characteristics Considered Desirable

Any rating device is necessarily an attempt to evaluate how good or how poor something is - whether the thing being rated is relatively "sufficient" or "deficient." Which approach is used does not really make too much difference, but the "sufficiency" approach seems more widespread, and has been utilized here. The "fully sufficient" intersection will, therefore, be rated 100 percent, and all others will be rated lower. The poorer the intersection, the lower the rating according to the relation to be established.

When the Traffic Rating is high (average delays are low) the Intersection Rating should be high. In general this condition will occur at low volumes where physical deficiencies are relatively unimportant, so the Physical Rating should have a limited effect on the Intersection Rating. The Intersection Rating should also reflect the increase in importance of physical deficiencies with decreasing Traffic Ratings (higher flows.)

Lowest Intersection Ratings should occur when both the

Physical and Traffic Ratings are low. This means high volume-to-capacity ratio plus poor physical condition equals minimum rating, indicating a need for immediate attention.

When the Physical Rating is high, approaching its maximum value of 70, the Intersection Rating should depend virtually entirely on the Traffic Rating. Hence, a good Physical Rating coupled with moderate flow at low delays should yield a high score, but the same good Physical Rating coupled with high flow with high delays should result in a low Intersection Rating.

Relation Adopted between Physical and Traffic Ratings

The relation adopted for the Intersection Rating basically begins with a perfect score of 170 and deducts for deficiencies found. Traffic Rating deficiencies are at all times fully deductible. Physical Rating deficiencies, however, are multiplied by a factor before being subtracted. This factor is a function of the Traffic Rating, such that if the Traffic Rating is zero (delays are high) the Physical Rating deficiency is also fully deductible. If, however, traffic is light and the Traffic Rating is one hundred percent, then only half of the Physical Rating deficiency is deductible.

In its basic form, the relation is:

$$IR = \frac{170 - (100-TR) - (70-PR) \left(\frac{100-0.5 TR}{100} \right)}{170} \times 100$$

What is left of the original 170 points after the fully-

deductible Traffic Rating deficiency (difference between perfect and actual ratings) and the variable Physical Rating deficiency deductions are applied, is divided by 170 points and multiplied by 100 so that the Intersection Rating is in percent.

Examination using limiting values for PR and TR showed that this relation conformed to the desirable boundary conditions established in the preceding section.

Rating Tables

As originally given, the equation is not convenient for calculation; the form given below, obtained by multiplication and collection of terms, was found much handier.

$$IR = \frac{270 TR + 200 PR - TR \times PR}{340}$$

For each level of one variable, using successive increments of the other, IR was systematically evaluated. Table 15 gives values of the Intersection Rating based on the above formula where both the Traffic Rating and the Physical Rating vary from zero to their maxima of 100 and 70, respectively, in increments of two. Intervening values not tabulated may be obtained by interpolation if necessary.

Table 15a

The Intersection Rating
(For Traffic Ratings of 80 to 100)

Physical Rating	Traffic Rating										
	100	98	96	94	92	90	88	86	84	82	80
70	100	99	98	96	95	94	93	92	91	89	88
68	99	98	97	96	95	93	92	91	90	89	88
66	99	98	96	95	94	93	92	90	89	88	87
64	98	97	96	95	93	92	91	90	89	87	86
62	98	96	95	94	93	92	90	89	88	87	85
60	97	96	95	93	92	91	90	88	87	86	85
58	96	95	94	93	91	90	89	88	86	85	84
56	96	95	93	92	91	90	88	87	86	85	83
54	95	94	93	91	90	89	88	86	85	84	83
52	95	93	92	91	90	88	87	86	84	83	82
50	94	93	92	90	89	88	86	85	84	82	81
48	94	92	91	90	88	87	86	84	83	82	80
46	93	92	90	89	88	86	85	84	82	81	80
44	92	91	90	88	87	86	84	83	82	80	79
42	92	90	89	88	86	85	84	82	81	80	78
40	91	90	88	87	86	84	83	82	80	79	78
38	91	89	88	86	85	84	82	81	80	78	77
36	90	89	87	86	84	83	82	80	79	78	76
34	89	88	87	85	84	82	81	80	78	77	76
32	89	87	86	85	83	82	80	79	78	76	75
30	88	87	85	84	83	81	80	78	77	76	74
28	88	86	85	83	82	81	79	78	76	75	73
26	87	86	84	83	81	80	78	77	76	74	73
24	86	85	84	82	81	79	78	76	75	73	72
22	86	84	83	82	80	79	77	76	74	73	71
20	85	84	82	81	79	78	76	75	74	72	71
18	85	83	82	80	79	77	76	74	73	71	70
16	84	83	81	80	78	77	75	74	72	71	69
14	84	82	81	79	78	76	74	73	71	70	68
12	83	81	80	78	77	75	74	72	71	69	68
10	82	81	79	78	76	75	73	72	70	69	67
8	82	80	79	77	76	74	73	71	69	68	66
6	81	80	78	77	75	73	72	70	69	67	66
4	81	79	77	76	74	73	71	70	68	67	65
2	80	78	77	75	74	72	71	69	67	66	64
0	79	78	76	75	73	71	70	68	67	65	64

Table 15b
The Intersection Rating
(For Traffic Ratings of 60 to 80)

Physical Rating	Traffic Rating										
	80	78	76	74	72	70	68	66	64	62	60
70	88	87	86	85	84	82	81	80	79	78	76
68	88	86	85	84	83	82	80	79	78	77	76
66	87	86	84	83	82	81	80	78	77	76	75
64	86	85	84	82	81	80	79	78	76	75	74
62	85	84	83	82	81	79	78	77	76	74	73
60	85	83	82	81	80	79	77	76	75	74	72
58	84	83	82	80	79	78	77	75	74	73	72
56	83	82	81	80	78	77	76	74	73	72	71
54	83	81	80	79	78	76	75	74	72	71	70
52	82	81	79	78	77	75	74	73	72	70	69
50	81	80	79	77	76	75	73	72	71	70	68
48	80	79	78	77	75	74	73	71	70	69	67
46	80	78	77	76	74	73	72	71	69	68	67
44	79	78	76	75	74	72	71	70	68	67	66
42	78	77	76	74	73	72	70	69	68	66	65
40	78	76	75	74	72	71	70	68	67	65	64
38	77	76	74	73	71	70	69	67	66	65	63
36	76	75	73	72	71	69	68	67	65	64	62
34	76	74	73	71	70	69	67	66	64	63	62
32	75	73	72	71	69	68	66	65	64	62	61
30	74	73	71	70	68	67	66	64	63	61	60
28	73	72	71	69	68	66	65	63	62	61	59
26	73	71	70	68	67	66	64	63	61	60	58
24	72	71	69	68	66	65	63	62	60	59	58
22	71	70	68	67	65	64	63	61	60	58	57
20	71	69	68	66	65	63	62	60	59	57	56
18	70	68	67	65	64	62	61	60	58	57	55
16	69	68	66	65	63	62	60	59	57	56	54
14	68	67	65	64	62	61	59	58	56	55	53
12	68	66	65	63	62	60	59	57	56	54	53
10	67	66	64	62	61	59	58	56	55	53	52
8	66	65	63	62	60	59	57	56	54	52	51
6	66	64	63	61	59	58	56	55	53	52	50
4	65	63	62	60	59	57	56	54	52	51	49
2	64	63	61	60	58	56	55	53	52	50	48
0	64	62	60	59	57	56	54	52	51	49	48

Table 15c
The Intersection Rating
(For Traffic Ratings of 40 to 60)

Physical Rating	Traffic Rating										
	60	58	56	54	52	50	48	46	44	42	40
70	76	75	74	73	72	71	69	68	67	66	65
68	76	74	73	72	71	70	69	67	66	65	64
66	75	74	72	71	70	69	68	66	65	64	63
64	74	73	72	70	69	68	67	66	64	63	62
62	73	72	71	70	68	67	66	65	63	62	61
60	72	71	70	69	67	66	65	64	62	61	60
58	72	70	69	68	67	65	64	63	62	60	59
56	71	69	68	67	66	64	63	62	61	59	58
54	70	69	67	66	65	64	62	61	60	58	57
52	69	68	66	65	64	63	61	60	59	58	56
50	68	67	66	64	63	62	60	59	58	57	55
48	67	66	65	63	62	61	60	58	57	56	54
46	67	65	64	63	61	60	59	57	56	55	53
44	66	64	63	62	60	59	58	56	55	54	52
42	65	64	62	61	60	58	57	56	54	53	52
40	64	63	61	60	59	57	56	55	53	52	51
38	63	62	61	59	58	56	55	54	52	51	50
36	62	61	60	58	57	56	54	53	51	50	49
34	62	60	59	57	56	55	53	52	51	49	48
32	61	59	58	57	55	54	52	51	50	48	47
30	60	59	57	56	54	53	52	50	49	47	46
28	59	58	56	55	53	52	51	49	48	46	45
26	58	57	55	54	53	51	50	48	47	45	44
24	58	56	55	53	52	50	49	47	46	45	43
22	57	55	54	52	51	49	48	46	45	44	42
20	56	54	53	51	50	49	47	46	44	43	41
18	55	54	52	51	49	48	46	45	43	42	40
16	54	53	51	50	48	47	45	44	42	41	39
14	53	52	50	49	47	46	44	43	41	40	38
12	53	51	50	48	47	45	43	42	40	39	37
10	52	50	49	47	46	44	43	41	40	38	36
8	51	49	48	46	45	43	42	40	39	37	36
6	50	49	47	45	44	42	41	39	38	36	35
4	49	48	46	45	43	41	40	38	37	35	34
2	48	47	45	44	42	41	39	37	36	34	33
0	48	46	44	43	41	40	38	37	35	33	32

Table 15d
The Intersection Rating
(For Traffic Ratings of 20 to 40)

Physical Rating	Traffic Rating										
	40	38	36	34	32	30	28	26	24	22	20
70	65	64	62	61	60	59	58	56	55	54	53
68	64	63	61	60	59	58	57	55	54	53	52
66	63	62	60	59	58	57	56	54	53	52	51
64	62	61	59	58	57	56	55	53	52	51	50
62	61	60	58	57	56	55	54	52	51	50	49
60	60	59	58	56	55	54	53	51	50	49	48
58	59	58	57	55	54	53	52	50	49	48	47
56	58	57	56	54	53	52	51	49	48	47	46
54	57	56	55	53	52	51	50	48	47	46	44
52	56	55	54	52	51	50	49	47	46	45	43
50	55	54	53	51	50	49	48	46	45	44	42
48	54	53	52	50	49	48	47	45	44	43	41
46	53	52	51	49	48	47	46	44	43	42	40
44	52	51	50	48	47	46	44	43	42	41	39
42	52	50	49	48	46	45	43	42	41	39	38
40	51	49	48	47	45	44	42	41	40	38	37
38	50	48	47	46	44	43	41	40	39	37	36
36	49	47	46	45	43	42	40	39	38	36	35
34	48	46	45	44	42	41	39	38	37	35	34
32	47	45	44	43	41	40	38	37	36	34	33
30	46	44	43	42	40	39	37	36	35	33	32
28	45	44	42	41	39	38	36	35	34	32	31
26	44	43	41	40	38	37	35	34	33	31	30
24	43	42	40	39	37	36	34	33	31	30	29
22	42	41	39	38	36	35	33	32	30	29	28
20	41	40	38	37	35	34	32	31	29	28	26
18	40	39	37	36	34	33	31	30	28	27	25
16	39	38	36	35	33	32	30	29	27	26	24
14	38	37	35	34	32	31	29	28	26	25	23
12	37	36	34	33	31	30	28	27	25	24	22
10	36	35	33	32	30	29	27	26	24	23	21
8	36	34	32	31	29	28	26	25	23	22	20
6	35	33	31	30	28	27	25	24	22	21	19
4	34	32	31	29	27	26	24	23	21	20	18
2	33	31	30	28	26	25	23	22	20	19	17
0	32	30	29	27	25	24	22	21	19	17	16

Table 15e
The Intersection Rating
(For Traffic Ratings of 0 to 20)

Physical Rating	Traffic Rating										
	20	18	16	14	12	10	8	6	4	2	0
70	53	52	51	49	48	47	46	45	44	42	41
68	52	51	50	48	47	46	45	44	42	41	40
66	51	50	48	47	46	45	44	42	41	40	39
64	50	49	47	46	45	44	42	41	40	39	38
62	49	47	46	45	44	43	41	40	39	38	36
60	48	46	45	44	43	41	40	39	38	37	35
58	47	45	44	43	42	40	39	38	37	35	34
56	46	44	43	42	40	39	38	37	35	34	33
54	44	43	42	41	39	38	37	36	34	33	32
52	43	42	41	40	38	37	36	34	33	32	31
50	42	41	40	38	37	36	35	33	32	31	29
48	41	40	39	37	36	35	33	32	31	30	28
46	40	39	38	36	35	34	32	31	30	28	27
44	39	38	37	35	34	33	31	30	29	27	26
42	38	37	35	34	33	31	30	29	27	26	25
40	37	36	34	33	32	30	29	28	26	25	24
38	36	35	33	32	31	29	28	26	25	24	22
36	35	34	32	31	29	28	27	25	24	23	21
34	34	32	31	30	28	27	26	24	23	21	20
32	33	31	30	29	27	26	24	23	22	20	19
30	32	30	29	28	26	25	23	22	20	19	18
28	31	29	28	26	25	24	22	21	19	18	16
26	30	28	27	25	24	22	21	20	18	17	15
24	29	27	26	24	23	21	20	18	17	16	14
22	28	26	25	23	22	20	19	17	16	14	13
20	26	25	24	22	21	19	18	16	15	13	12
18	25	24	22	21	19	18	17	15	14	12	11
16	24	23	21	20	18	17	15	14	12	11	9
14	23	22	20	19	17	16	14	13	11	10	8
12	22	21	19	18	16	15	13	12	10	9	7
10	21	20	18	17	15	14	12	10	9	7	6
8	20	19	17	15	14	12	11	9	8	6	5
6	19	18	16	14	13	11	10	8	7	5	4
4	18	16	15	13	12	10	9	7	5	4	2
2	17	15	14	12	11	9	7	6	4	3	1
0	16	14	13	11	10	8	6	5	3	2	0

Field Validation Investigation

As mentioned earlier, sufficiency rating systems must possess validity and discrimination to be of any value. To determine whether or not the rating procedure outlined possessed these characteristics it was considered desirable to conduct a field investigation. Five intersections were selected for study. All were controlled by fixed-time signals, and all approaches were sufficiently removed from adjacent signals that arrivals could be considered to comply with the assumptions previously stated. Normal two-phase control was utilized on all approaches except one, where a leading left-turn arrow was in use. Otherwise, a wide range of design features existed, including skew, added lanes, lane-use marking and variable width streets. Vehicular and pedestrian volumes also were found to vary widely.

Ratings Applied

After the intersections to be studied were selected each was rated using a word description and the intersections were ranked in order of sufficiency based on judgment as to relative performance. These ratings were made by the author.

After the "judgment ratings" were made, the previously-outlined Intersection Rating procedure was utilized. The various Traffic Rating components of the Intersection Ratings were based on volumes during evening peak hours. Volumes were recorded by fifteen-minute periods from 4:30 until 5:30 PM. The peak fifteen-minute flows were selected for the

various approaches, and these volumes were multiplied by four to give the critical volumes on an hourly basis for conversion to standard lane volumes. Commercial vehicle and turns percentages were also obtained on a fifteen-minute basis, but average values over the entire hour were used for the capacity calculations. These calculations were made in accordance with the procedures set forth in the Highway Capacity Manual (18, 68). The rating charts (Figures 17 through 25) were used to determine the approach Traffic Ratings, and the composite Traffic Ratings for the intersections were then computed in accordance with the procedure mentioned earlier.

To determine the Physical Rating components of the Intersection Ratings, rather complete sketch maps of the intersections were made. Use of a Rolatape (distance-measuring wheel) facilitated this work considerably. Street and lane widths, curb radii and distances to view obstructions were obtained in the field with the Rolatape and notes were made on pavement and surface characteristics. All intersections were known to have been originally paved with Portland cement concrete; four of the five had been resurfaced with bituminous concrete. Where one view obstruction was obviously critical, its location was measured. Where several obstructions appeared to be equally critical, all were investigated.

Actual intersection areas (as defined earlier) were carefully planimetered from scale drawings of the respective

intersections.

An illumination reading was taken at the center of each intersection approach where such approach intersected the crosswalk line which was closer to the intersection. The Illumination rating for each intersection was then based on the average of all its approach illumination readings. A Weston Illumination Meter, Model 603 was used. The separate "light paddle" (which contains the photocells) was laid flat on the street surface at the specified points, and the separate meter was read several feet away to avoid masking any light source.

Comparison of Rating Results

Table 16 shows the results of applying the Intersection Rating method to the five intersections selected, and compares these objective results with the "judgment ratings" made earlier. Table 16 lists the intersections in order of sufficiency as determined by the Intersection Rating. It will be observed that, in this case, the ranking by Traffic Ratings is the same as the ranking by Intersection Ratings, and that the respective values are very close to each other. It will further be observed that the intersection rated as Good by the subjective rating also rated highest objectively, that those intersections subjectively rated as Average were rated next in order by the objective method although the order of the two intersections was reversed, and similarly that the two Poor intersections were rated lowest by the objective

Table 16

Comparison of Objective and Subjective Ratings for Five Intersections

<u>Intersection</u>	<u>Subjective Rating</u>		<u>Objective Rating</u>			
	<u>Description</u>	<u>Rank</u>	<u>Physical Rating</u>	<u>Traffic Rating</u>	<u>Intersection Rating</u>	<u>T.R. Rank</u> <u>I.R. Rank</u>
Fourth & Owen	Good	1	51	94	91	1 1
Grant & Northwestern	Average	3	44	88	84	2 2
Main & Kossuth	Average	2	37	86	81	3 3
Grant & State	Poor	5	45	78	78	4 4
ByPass 52 & South	Poor	4	38	61	64	5 5

method with their order also interchanged from that of the subjective rating.

The Intersection Ratings for the two intersections subjectively rated Average were nearly the same, and the change of order by the objective rating is not unreasonable. For the two Poor intersections, the two Intersection Ratings differed appreciably. In this case it is felt that the subjective ratings were in error, as experience indicates to be common. Critical evaluation of the delay at both intersections clearly indicated that congestion and delay were greater at the By Pass 52 and South Street intersection because of inequitable division of the signal cycle at the peak hour. At this intersection the phase and cycle lengths remained the same throughout the day while at the State and Grant Street intersection a two-dial controller permitted the use of different phase and cycle lengths during peak and non-peak hour conditions. It is believed that the inversion of the rating order by the objective method for these two intersections is justified.

SUMMARY AND CONCLUSIONS

A general procedure has been established whereby the sufficiency of any urban intersection may be determined. The sufficiency of an intersection is evaluated by rating the physical and traffic characteristics with major emphasis placed on the ability of the intersection to handle the required traffic movements. Average delay is used as the important measure of this ability. Specific evaluation data are developed for those intersections where control is by fixed-time signal and uniform arrival may be assumed.

Field investigations confirmed that average vehicular delays which occur at intersections can be estimated by the solution of mathematical models which include consideration of intersection capacity, approach volume, intersection control characteristics, and vehicular spacing when entering the intersection. Further field investigation indicated that the over-all rating procedure produces reasonable results and that it discriminates among intersections whose characteristics are nearly the same.

The rating procedure presented in this report is recommended for evaluating the sufficiency of urban intersections. The sufficiency estimate thus obtained should be used as a tool in connection with other pertinent considerations to establish improvement priorities.

RECOMMENDATIONS FOR FURTHER RESEARCH

Any intersection may be rated using the procedure outlined in this report by combining its Physical Rating and its composite Traffic Rating to arrive at a percentage score for the Intersection Rating. The Physical Rating as outlined may be applied to any urban intersection. The fundamental concept of the Traffic Rating, namely that the rating value is based on the average delay incurred by users, is likewise universally applicable. In addition, this report has developed detailed relations between average delay, traffic volume and signal characteristics that are applicable in certain cases felt to be critical. It is suggested that further research be undertaken to develop similar delay-volume-control relations under other conditions. Specifically, three areas are proposed. First, in the case of fixed-time signal control, research should be done on the situation where arrival may not be considered uniform, as in the case of "platoon" movement of vehicles due to a preceding signal or other cause. Second, an increasing need will be felt for similar delay-volume-control relations where control is by actuated signals of either the semi- or fully-actuated type. Investigation of this condition is advocated. Third, many intersections with stop sign control are troublesome, and study of the volume-delay relations at such locations is

recommended.

Inasmuch as the successive intersection entry times reported by Greenshields, Shapiro and Ericksen (26) are based on a limited study, further investigation is desirable to refine these values. In the same paper, the action of "close decision" cars confronted by the amber clearance light was reported. The number of cars studied was relatively small, however; it is recommended that further information be obtained on the behavior of such "close decision" cars, utilizing a larger number of observations.

Standard lane volumes below two hundred cars per hour were arbitrarily considered too small for the uniform-arrival assumption to hold. Investigation to determine a more precise cut-off value is suggested.

It is felt that further research would be of value in the cases where approaches operate at above-capacity volumes. Only a limited number of such data points were obtained in the field investigation conducted in connection with the Traffic Rating validation, and more information on operations above capacity is considered desirable to better study the agreement between the theory and actual operations in these regions.

Finally, while average delay may be calculated (on the basis of the stated assumptions) for below-capacity volumes and a specific answer obtained, this is not the case for above-capacity operation where the answer obtained depends

upon the delay definition adopted. It is suggested that other definitions of average delay in the above-capacity region be established, and that a study be made of the effects of these different definitions on the shapes of the above-capacity portions of the volume-delay curves.

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APPENDIX

Theoretical Volume-Delay Curves

FIGURE 42. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 40 SECOND CYCLE.

FIGURE 43. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 45 SECOND CYCLE.

FIGURE 44. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 50 SECOND CYCLE.

FIGURE 45. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 55 SECOND CYCLE.

FIGURE 46. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 60 SECOND CYCLE.

FIGURE 47. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 65 SECOND CYCLE.

FIGURE 48. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 70 SECOND CYCLE.

FIGURE 49. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 75 SECOND CYCLE.

FIGURE 50. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 80 SECOND CYCLE.

VITA

VITA

William Walter Schenler was born in Decatur, Illinois, on September 1, 1923. The following year, his family moved to Maplewood, Missouri, (a suburb of Saint Louis) where he still resides. He was educated in the Maplewood-Richmond Heights school system, and was graduated as class co-valedictorian from Maplewood-Richmond Heights High School in June 1941. In September 1941 he entered Purdue University, where he completed three academic years before being called to active duty in the army in World War II. He entered military duty as a private, served for three years and was returned to inactive duty in 1946 as a first lieutenant. He continued active in the military reserve and presently holds the rank of major. In the fall of 1946 he returned to Purdue and was graduated in June 1947 with the BSCE degree. From September 1947 until June 1951 he taught Civil Engineering at Purdue, holding the rank of Instructor. Concurrently he worked toward the degree of MSCE in Structures, which was received in June 1950. In June 1951, Mr. Schenler entered the employ of the Ben Hur Construction Company, a steel construction firm at Saint Louis, where he served as structural engineer and field superintendent. In September 1954 he joined the faculty of Washington University at Saint Louis as Assistant Professor of Civil Engineering.

Doctoral work, begun in June 1956, was carried on through the summer of 1957 under fellowships granted by the Automotive

Safety Foundation while Mr. Schenler was on a one-year leave from Washington University. During 1959-1960 he was again on leave from Washington University, and completed the Ph.D. research and thesis requirements at Purdue, holding a Graduate Assistantship from Purdue and a fellowship grant from the Esso Foundation through Washington University. Upon leaving Purdue, he will resume his staff duties at Washington University. Mr. Schenler is a Registered Professional Engineer in Missouri and is a member of the following professional societies: Tau Beta Pi, Chi Epsilon, American Society of Civil Engineers, American Society for Engineering Education and The Society of the Sigma Xi.

In 1951 he was married to Carolyn Joan Schroeder Schenler, and they have two children. The family is active in the work of Christ Church, United Church of Christ, Maplewood, Missouri.