Interchange Versus At-Grade Intersection on Rural Expressways

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The economic benefits and costs of replacing a two-way stop-controlled intersection on a rural expressway with either a signalized intersection or a conventional diamond interchange were compared. Economic benefits were based on the difference in road user costs among alternatives. Road user costs were composed of five components: delay, idle fuel, acceleration-deceleration delay, speed-change running costs, and accident costs. The benefit-cost analysis of the signalized intersection and interchange under rural expressway conditions indicated that the interchange was a more economically viable alternative than the signalized intersection. The signalized intersection's main benefit is a reduction in accidents; however, this benefit is generally negated by the signal's higher operational costs whenever the minor road demand is less than one-half that of the major road. Three geometric scenarios were formulated for the intersection and interchange. The first considered a four-leg junction with a two-lane minor road. The second considered a four-leg junction with a four-lane minor road. The third considered a three-leg junction with a two-lane minor road. Three figures were developed relating the major and minor road daily traffic demands that would economically justify an interchange in terms of a benefit-cost ratio. Whenever the major road demand is about 4,000 vehicles per day (vpd) or more, the minor road demands that provide a 2.0 benefit-cost ratio are about 4,000, 6,500, and 8,000 vpd for the three scenarios, respectively.

The ability to assess the cost-effectiveness of a construction project is particularly important when an agency is operating under a limited construction budget. Some state agencies have adopted general procedures and conditions under which an interchange is warranted; however, these criteria do not typically consider accident history or road user costs. An economic analysis of road user benefits (e.g., reduced delay, fuel, stops, and accidents) and project costs would justify a project from a pragmatic standpoint and could facilitate alternative selection and project prioritization.

The objective of this research was to develop guidelines for use in determining when a signalized intersection or an interchange would be a cost-effective corrective measure for a problematic unsignalized intersection on a rural expressway. Such an intersection might be experiencing operational or safety problems. The guidelines described in this paper are based on an economic assessment of the operational and safety benefits realized by upgrading a two-way stop-controlled intersection to a signalized intersection and to a conventional diamond interchange for a range of traffic demand levels.

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LITERATURE REVIEW

Interchanges are typically justified for one of two reasons. An interchange may be necessary to maintain a consistency with the major road's functional classification. Alternatively, an interchange may be needed to efficiently serve existing or future traffic demands. The general trend among highway departments is to deal with candidate interchange locations on a case-by-case basis.

A recent survey of state departments of transportation by the Nebraska Department of Roads (NDOR) (*1*) indicated that most states do not have policies or guidelines that identify the conditions needed to justify an interchange. Many states indicated that the decision to develop a grade separation or interchange is a direct consequence of the decision to provide an access-controlled roadway.

Two of the states that responded to the survey indicated that they design all new rural expressways that bypass cities as full-access control facilities. In this regard, they construct interchanges at all major intersections along the bypass. The justification offered for this policy was the poor safety record of existing bypasses with at-grade intersections.

One research effort at establishing interchange warrants was conducted by Ockert and Walker (*2*). On the basis of a series of simulation studies for a range of economic and traffic characteristics, they concluded that the operational benefits of an interchange begin to outweigh its cost at traffic volume levels that meet or exceed the *Manual on Uniform Traffic Control Devices*' (MUTCD) (*3*) Signal Warrants 1 and 2 (Warrants 9 and 11 did not exist at the time of this study). In other words, the benefit-cost ratio of the interchange is higher than that of a conversion to signal control at demand levels that meet MUTCD signal warrants, although both ratios are greater than 1.0. This implies that signal control is not an economically justifiable alternative on rural expressways.

Another effort at establishing interchange warrants was conducted by Van Every (*4*). Van Every calculated the road user benefits for a conventional diamond interchange compared with a two-way stop-controlled intersection. He then compared these benefits with the incremental cost of constructing an interchange and made recommendations on major and minor road volume thresholds that would yield benefit-cost ratios of 1.0 and 2.0. Van Every (*4*) reported volume warrants for several combinations of traffic growth rates and turn percentages. Examination of these warrants indicates a strong similarity to MUTCD Signal Warrant 11 [64 km/hr (40-mph) major road speed], which is consistent with the findings of Ockert and Walker (*2*).

ANALYSIS METHODOLOGY

The analysis methodology was based on the benefit-cost comparison of three design alternatives. The base alternative was defined as the two-way stop-controlled intersection. The two alternatives included a signalized intersection and a conventional diamond interchange with stop-controlled off ramps. The benefit-cost approach used in this research was based on the methods described in *A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements* (i.e., Red Book) (*5*).

To assess the relative benefits of the three design alternatives, procedures for quantifying the operational and safety performance of the typical intersection and diamond interchange were established. The operational assessment procedure is sensitive to a variety of design factors, such as traffic demand, traffic composition, design speed, and traffic control. This procedure is based on the quantification of various measures of motorist cost, such as motorist delay and vehicle fuel consumption. The assessment of operational performance is consistent with the methodology published in the 1985 *Highway Capacity Manual* (*6*) (i.e., 1985 HCM).

The safety assessment procedure developed for this research is sensitive to traffic demands, traffic control type, and junction type. This procedure was developed using data from the Highway Safety Information System (HSIS) (*7*). The procedure is composed of several regression models for predicting the expected frequency and severity of accidents at a junction on the basis of its traffic demand and traffic control conditions.

The attractiveness of each design alternative was assessed by comparing the present worth of its benefits and costs with the base alternative. Costs considered for this analysis included construction cost, maintenance cost, and residual worth (a negative cost) of the property at the end of the alternative's design life.

Economic Factors

Discount Rate

The discount rate is needed to calculate the worth of all benefits and costs in terms of present dollars. The discount rate used in this analysis is based on the market rate of return adjusted for inflationary effects (i.e., a "constant dollar" approach) and is assumed to be 4.0 percent, based on a market rate of return of about 9.2 percent and a rate of inflation of 5.0 percent.

Cost Updating Procedure

Dollar values for all unit prices and project cost components have been updated to January 1991 levels using appropriate consumer and wholesale price indices. All of the unit prices used in this study were extracted from the Red Book (*5*) with the exception of accident costs, which were obtained from FHWA (*8*).

Value of Travel Time

For this study, travel time rates for passenger cars, single-unit trucks, and tractor-trailer trucks were obtained from the Red Book (*5*,pp.15–19). All values were updated to January 1991 levels. The updated value of time for passenger cars was estimated as $0.84/hr. This estimate is based on an "average" value for all trip types (e.g., social and work), a relatively low time savings due to any operational improvements (less than 5 min), and an average vehicle occupancy of 1.56 persons. The updated value of time for single-unit and tractor-trailer trucks is estimated as $15.50/hr and $17.72/hr, respectively.

Analysis Period

For this study, the lifetimes of the geometric improvements associated with the alternative designs are expected to range from 20 to 40 years. However, because of the uncertainties in the predictions of future traffic demands, travel patterns, and land use, the duration of the analysis period was established as 20 years. This duration was believed to be more defensible in terms of our greater confidence in analysis assumptions made for shorter periods. A 20-year period was also believed to yield more conservative results by making the alternative justify its entire construction cost with benefits accrued over the first 20 years.

Traffic Characteristics

Directional Distribution

The directional distribution of traffic demand on each junction approach was established as 50 percent inbound and 50 percent outbound.

Annual Growth Rate

NDOR's *1990 State Highway Plan and Highway Needs Report* (*9*) was consulted to determine a range of typical growth rates on rural Nebraska highways. Present and future average daily traffic values (ADTs) were selected from this document for 19 segments of six state highways. These ADTs were used to calculate the annual growth rate for each segment. The calculated growth rates were found to range from 2.4 to 4.6 percent per year. The average rate of 3.5 percent per year was established as a typical value.

Turn Percentages

Turn movement percentages were obtained for 24 rural intersections where at least one of the intersecting routes was a state highway. An examination of these turn percentages indicated a relationship between turn percentage, traffic pattern, and demand levels. To accurately model this relationship for the full range of demand conditions, an algorithm for predicting turn percentages was incorporated into the benefit-cost analysis program. This algorithm was developed by Hauer et al. (*10*) but enhanced to calculate the "balanced" approach and departure volumes for specified ADTs. By incorporating this algorithm in the analysis program, all that was needed as input was an estimate of the major and minor road ADT; hourly turn movement volumes could then be estimated by the algorithm.

Application of the algorithm to the 24 intersections, using initial "seed" turn percentages of 10 percent left and right on the major road and 30 percent left and right on the minor road, indicated that it could predict the actual turn percentages reasonably well. In fact, the algorithm was able to explain 87 to 92 percent of the variation in turn percentages on the major road and 61 to 68 percent of the variation on the minor road.

Traffic Composition

NDOR's 1990 report (*9*) was consulted to determine the percentage of truck traffic on rural Nebraska highways. The percentage of trucks in the daily traffic stream was selected from this document for 19 segments on six state highways. These percentages were found to range from 6 to 25 percent, with a representative value established as 14 percent (7 percent single-unit trucks and 7 percent tractor-trailers).

Hourly Volume Frequencies

The analysis program was written to estimate user costs on an hourly basis. However, instead of analyzing all 8,760 hr in each year of the analysis period, only five representative hourly volumes were considered for each year. Each hourly volume level is estimated by multiplying an hourly volume factor by the ADT for the corresponding year. The total user costs for each year are then calculated as the sum of the user costs for each of the five hourly volumes multiplied by the corresponding number of hours represented. This process is then repeated for each analysis year.

The five hourly factors used by the program were developed by first ranking and then dividing the frequency distribution of hourly flows for 1 year into five intervals and calculating an average hourly volume for each interval. The frequency distribution was obtained from the hourly volumes recorded at seven automatic traffic recorder (ATR) stations on rural highways in Nebraska (*11*). The interval widths, in hours, were selected to include hourly flows of similar magnitude. As a result, the interval widths were small for the few peak hours and larger for the many nonpeak hours.

Five hourly factors were calculated for each ATR station for the same hour intervals. Each factor was calculated by dividing the average hourly volume for the corresponding interval by the ADT. These factors were then averaged over all seven stations. The resulting hourly volume factors (expressed as a percentage of ADT) and associated hour intervals are 9.5 percent for 250 hr, 8.6 percent for 250 hr, 7.6 percent for 1,000 hr, 5.9 percent for 2,500 hr, and 1.6 percent for 4,760 hr.

Average Travel Speed

The average travel speed was estimated as 88 percent of the 85th percentile speed on the basis of information provided elsewhere (*12*). The 85th percentile speed was, in turn, assumed to equal the posted speed limit. The posted speed limits on the major and minor roads were assumed to be 89 and 64 km/hr (55 and 40 mph), respectively. The ramp speed for the conventional diamond interchange was assumed to be 72 km/hr (45 mph). The right-turn speed was assumed to be 21 km/hr (13 mph). All left-turning vehicles were assumed to stop.

Project Characteristics

Geometric Configuration

The geometry of the intersection and interchange included the following elements:

- A four-lane expressway/major road with a 12-m (40-ft) median.
- The signalized and unsignalized intersections have left-turn bays on the major road. Only the signalized intersection has left-turn bays on the minor road.
- The on- and off-ramps for the interchange intersect with the major road about 549 and 488 m (1,800 and 1,600 ft) from the minor road, respectively. The ramp/minor road junctions are offset 183 m (600 ft) from the major road centerline.
- The major road is constructed at grade through the interchange and the minor road is elevated above the major road. The bridge is 61 m (200 ft) in length, supported by a bent in the major road median.

Construction Costs

For this study, cost estimates were formulated for each of the three junction types. The construction cost for the unsignalized intersection includes only the costs of upgrading the minor road approaches and adding left-turn bays on the major road. The construction cost for the signalized intersection includes upgrading and widening the minor road approaches, providing left-turn bays on all approaches, and installing a traffic signal. Two cost estimates were made for the interchange: one for a two-lane overpass and one for a four-lane overpass. The cost estimates included costs for mobilization, earthwork, subgrade preparation, pavement, and drainage.

The following assumptions are included in the cost estimates: (a) the cost of the four-land major road is common to all junction types and can be excluded; (b) contingency costs would amount to 20 percent of the construction cost; and (c) the cost of engineering is 14 percent of the contingency and construction costs. The estimated 1991 construction costs for the alternatives considered in this study are as follows: unsignalized intersection, $156,000; signalized intersection, $363,000; conventional diamond interchange (two-lane), $2,020,000; and conventional diamond interchange (four-lane), $3,060,000.

Maintenance Costs

Estimates of annual maintenance costs for 1991 were obtained from NDOR. These data indicate annual maintenance costs of \$2,000 for unsignalized intersections and \$7,000 for interchanges with two-lane minor roads. Maintenance costs for interchanges with four-lane minor roads were not obtained; however, these costs were assumed to be \$10,000 per year.

The annual maintenance cost for a signalized intersection was estimated to be about \$1,000 higher than that of the unsignalized intersection; the annual operating cost of the signal is \$1,000. Thus, the operating and maintenance costs of the signalized intersection were estimated at \$4,000 per year.

Residual Value

The residual value of the alternative facilities was not included in the analysis. This omission resulted in a more conservative analysis in that the alternative would have to justify its total construction cost in terms of road user benefits.

Operational Costs

Time Cost Components

Time cost components include all traffic control or geometric factors that delay motorists. These costs represent the excess travel time (i.e., delay) incurred by motorists because of the junction. Thus, they represent the difference between the actual travel time and the travel time that would have been incurred had all movements been served by exclusive through lanes or directional ramps. The most significant time cost components are

1. The delays from having to transition from the main lane speed to a turn speed and back to the main lane speed,
2. The delays to minor movements that are stop controlled, and
3. The delays to all movements that are signal controlled.

In those instances in which the predicted delays exceed reasonable values (e.g., 30 min), it is assumed that the drivers will actually divert to alternative routes or postpone their trips to other times. Thus, delays in excess of 30 min could conceptually represent a notional cost that reflects the added inconvenience of diversion or postponement. For this analysis, the predicted delays were limited to the duration of the period of analysis (i.e., 1 hr). The maximum delay of 1 hr reflects a reasonable upper limit to the concept of notional cost.

Excess Delay for Speed Change The excess delay for a speed change cycle is calculated using constant rates of acceleration and deceleration. The rates used are for normal operating conditions and for speeds up to 64 km/hr (40 mph). The rates for passenger cars and single-unit trucks are estimated as 5.3 km/hr/sec (3.3 mph/sec) for acceleration and 8.0 km/hr/sec (5.0 mph/sec) for deceleration (*13*). The rates for tractor-trailer trucks are estimated as 1.6 km/hr/sec (1.0 mph/sec) for acceleration and 6.4 km/hr/sec (4.0 mph/sec) for deceleration.

Excess delay for a speed change is calculated using the following equations:

$$t_{ex} = t_{ad} - t_{ff} \tag{1}$$

$$t_{ad} = \frac{v_1 - v_0}{d} + \frac{v_2 - v_1}{a} \tag{2}$$

$$x_t = \frac{v_1^2 - v_0^2}{2d} + \frac{v_2^2 - v_1^2}{2a} \tag{3}$$

$$z = \frac{v_2^2 - v_0^2}{2x_t} \tag{4}$$

$$t_{ff} = \begin{cases} x_t/v_0 & \text{if } z = 0 \\ (-v_0 + \sqrt{v_0^2 + 2zx_t})/z & \text{otherwise} \end{cases}$$

where

t_{ex} = excess delay for a speed change cycle (sec/vehicle),
t_{ad} = travel time for the acceleration-deceleration maneuver (sec/vehicle),
t_{ff} = travel time at free flow speed uninterrupted by junction (sec/vehicle),
v_0 = initial speed (mps),
v_1 = speed decelerated to (mps),
v_2 = speed accelerated back to (mps),
a = acceleration rate (mpss),
d = deceleration rate (a negative value) (mpss),
x_t = travel distance during deceleration and acceleration (m), and
z = average acceleration needed to transition from v_0 to v_2 (mpss).

Unsignalized Intersection Delay Average delay to a stop-controlled movement is estimated using the following equation derived by Tanner (*14*):

$$w = \frac{q_1 e^{\beta q_1}(e^{\alpha q_1} - \alpha q_1 - 1) + q_2 e^{\alpha q_1}(e^{\beta q_1} - \beta q_1 - 1)}{q_1[q_1 e^{\beta q_1} - q_2 e^{\alpha q_1}(e^{\beta q_1} - 1)]} \tag{5}$$

where

w = average delay to minor road vehicle (sec/vehicle),
q_1 = major road flow rate (vehicles/sec),
q_2 = minor road flow rate (vehicles/sec),
e = base of natural logarithms (2.7183 . . .),
α = critical acceptance gap (sec), and
β = follow-up gap (sec).

The length of the critical gap for passenger cars was taken to be 6.0 sec for the higher speed conditions found at rural expressway intersections (*6*). On the basis of the work of Fitzpatrick et al. (*15*), tractor-trailer trucks were assumed to have an 8.0-sec critical gap. The follow-up gaps for passenger cars and trucks are assumed to be 2.0 and 3.0 sec, respectively.

Signalized Intersection Delay Delays for signal-controlled movements associated with the signalized intersection alternative were calculated using the methodology described in the 1985 HCM (*6*). For this analysis, the procedures described in the 1985 HCM (Chapter 2 of Appendix II) were used to calculate the signal cycle length and phase splits. It was assumed that the intersection operated in a semiactuated mode with all excess time allocated to the major movements.

The delay to stopped vehicles is calculated using the following formulas:

$$d = \frac{d_1 + d_2}{1.3} \tag{6}$$

$$d_1 = \frac{C\left(1 - \frac{g}{C}\right)^2}{2\left(1 - \frac{g}{C}X\right)} \tag{7}$$

$$d_2 = 900\ TX^2\left((X - 1) + \sqrt{(X - 1)^2 + \frac{4X}{Tc}}\right) \tag{8}$$

where

- d = average stopped delay (sec/vehicle);
- d_1 = uniform arrival delay component (sec/vehicle);
- d_2 = random arrival delay component (sec/vehicle);
- C = cycle length (sec);
- T = duration of the period of analysis (1 hr) (hr);
- g = effective green time (sec);
- X = v/c ratio for movement;
- c = sg/C, capacity of the movement (vph); and
- s = saturation flow rate of the movement (assumed as 1,800 vphgpl) (vphg).

Running Cost Components

Excess Running Cost for Speed Change The running costs for a speed change cycle are presented in Tables B-10, B-11, and B-12 in the Red Book (*5*) for a range of speeds and vehicle types. Least-squares regression techniques were used to develop the following equations for predicting the trends shown in these tables:

$$R_p = -2.07 + 0.477S_i - 0.411S_r + 0.00222S_i^2 + 0.00397S_r^2 - 0.00674S_iS_r \tag{9}$$

$$R_{su} = -7.55 + 1.467S_i - 1.077S_r - 0.00196S_i^2 + 0.00133S_r^2 - 0.00401S_iS_r \tag{10}$$

$$R_{wb} = -27.33 + 4.578S_i - 3.355S_r + 0.00606S_i^2 + 0.0221S_r^2 - 0.0396S_iS_r \tag{11}$$

where

- R_p = passenger car speed change cost per 1,000 cycles (dollars),
- R_{su} = single-unit truck speed change cost per 1,000 cycles (dollars),
- R_{wb} = tractor-trailer truck speed change cost per 1,000 cycles (dollars),
- S_i = initial and final speed (mph) (1 mph = 1.61 km/hr), and
- S_r = speed reduced to and returned from (mph) (1 mph = 1.61 km/hr).

In those instances in which the initial and final speeds are not the same, their average value is used as an approximation for S_i.

These costs represent the running costs per 1,000 speed changes at January 1975 price levels. They were inflated to the 1991 (base) analysis year by multiplying by the ratio of the 1991 price indexes to those for 1975. These ratios are 2.52 for passenger cars and 2.21 for single-unit and tractor-trailer trucks.

Idling Costs at an Intersection Idling costs are primarily dependent on the composition of the stopped traffic queue. Trucks typically consume less fuel and oil while idling and thus have lower idling costs than passenger cars. The costs for 1,000 hr of idling were obtained from the Red Book (*5*). These costs at 1991 levels are \$790, \$613, and \$427 per 1,000 idling hr for passenger cars, single-unit trucks, and tractor-trailer trucks, respectively.

Accident Costs

Accident prediction models were developed for this research that are specific to junction type. The accident data base used to calibrate these models was obtained from FHWA (via the Highway Safety Research Center at the University of North Carolina). This data base was subset from HSIS (*7*) to contain only nonurban junctions. The key feature of the HSIS is its inclusion of roadway design and traffic volume data. This added information permits the investigation of cause-and-effect relationships between geometry and traffic demand and accident frequency and severity.

The approach taken in developing an accident prediction model for this study was based on procedures described by Hauer et al. (*16*). Hauer has argued against using traditional least-squares regression of accident data because of violations of several assumptions on which this type of analysis is based. Instead, Hauer advocates the use of a general linear model [e.g., GLIM (*17*)] wherein these assumptions are removed, thereby yielding a better predictor of accident frequency as influenced by other factors.

Several factors were examined for their effect on accidents, including junction type, average daily traffic demand, speed limit, and median width. Unfortunately, correlation analysis among these variables indicated strong interrelationships between traffic demand, speed, and median width, which precluded their combined use in one accident model. Because traffic demand had the strongest correlation with accident frequency, it was selected for inclusion in the final accident model. The models developed using this approach for two-way stop-controlled intersections, signalized intersections, and interchanges are given by Equations 12, 13, and 14, respectively.

$$E(m)_U = 0.6503 * \left(\frac{T_m}{1,000}\right)^{0.2925} * \left(\frac{T_c}{1,000}\right)^{0.7911} \quad (12)$$

$$E(m)_S = 0.3603 * \left(\frac{T_m}{1,000}\right)^{0.7213} * \left(\frac{T_c}{1,000}\right)^{0.3663} \quad (13)$$

$$E(m)_I = 0.04864 * \left(\frac{T_m}{1,000}\right)^{1.337} \quad (14)$$

where

$E(m)$ = expected number of accidents per year,
T_m = major road traffic demand (vehicles/day), and
T_c = minor (cross) road traffic demand (vehicles/day).

Accident costs were obtained from an FHWA technical advisory (*8*). The technical advisory provides estimates of accident costs as of 1986 and is based on a willingness-to-pay approach. This approach includes the direct and indirect costs associated with the accident as well as the amount the typical individual is willing to pay to avoid harm. The costs per incident recommended in this advisory are as follows: fatality, $1,500,000/person; injury (overall average), $11,000/person; and property damage only (PDO), $2,000/vehicle.

These costs were combined with accident severity data to predict average accident costs for each junction type. The property cost component of the average accident cost was calculated by first inflating the PDO cost by 50 percent. This inflation stems from the fact that there is an average of 1.5 vehicles per accident on rural roadways in Nebraska (*18*). As a result of these computations, the average accident costs (updated to 1991 price levels) were determined as $45,500 at two-way stop intersections, $23,000 at signalized intersections, and $19,800 at interchanges.

The accident cost at two-way stop-controlled intersections is about twice that of the other two junction types. This disparity stems from the greater severity of accidents found at this type of intersection (*19*).

Road User Benefit Analysis

Sensitivity Analysis

Several variables were initially considered in this examination of sensitivity to determine which had the greatest potential impact on user benefits. The variables found to have the most significant impact were minor road traffic demand and discount rate. To a lesser extent, major road demand, traffic composition, and the annual traffic growth rate also had some influence on the amount of user benefits. Because it was the most influential, minor road demand was included in the sensitivity analyses of the other variables.

The sensitivity of road user benefit to minor road demand and discount rate is shown in Figure 1. The benefits shown are those derived from the operation of an interchange instead of a two-way stop-controlled intersection. Major road demand was set at 10,000 vpd. In general, an increase in the discount rate significantly decreases the road user benefits. Alternatively, the minimum minor road traffic demand needed to achieve the level of user benefit that justifies an interchange

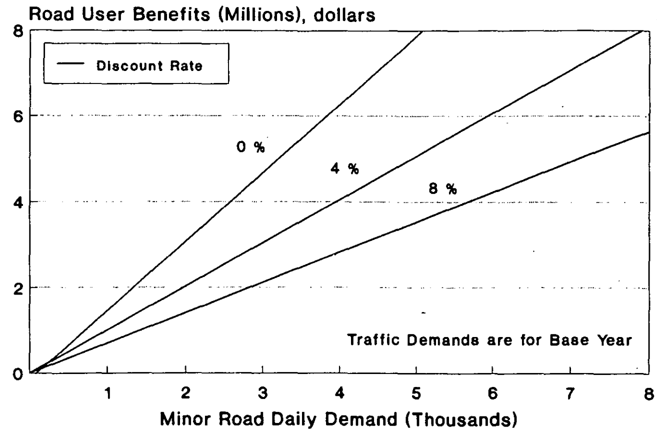

FIGURE 1 Road user benefits as affected by minor road demand and discount rate.

would be increased (on the basis of equating road user benefits to the incremental construction cost of the interchange alternative). As a result of this sensitivity, a common discount rate must be used when making comparative assessments of the economic worth of competing projects.

Road User Cost Components

Road user costs are composed of the operational and accident costs associated with the particular junction type. Operational costs can be further categorized as stopped delay, acceleration-deceleration delay, idling fuel consumption, and acceleration-deceleration running costs (e.g., fuel, brakes, and tires). The percentage contribution of each component to the total user cost is shown in Figure 2.

As this figure indicates, acceleration-deceleration running costs constitute the major contributor to user costs. These costs generally range from 40 to 75 percent of the total road user costs, depending on junction type. Accident costs are the next biggest contributor, ranging from 15 to 50 percent. In contrast, stopped delay, acceleration-deceleration delay, and idling fuel collectively do not contribute more than 20 percent to the road user costs.

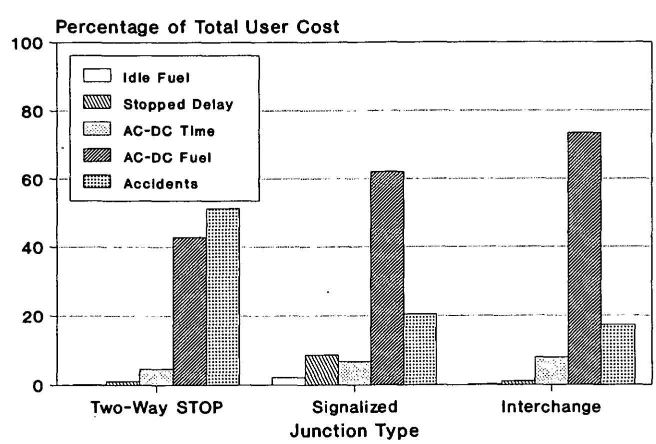

FIGURE 2 Typical road user cost components for each junction type.

INTERCHANGE GUIDELINES

Approach

The approach taken in developing the interchange guidelines is based on a benefit-cost (B/C) ratio analysis. This approach was preferred to a net present value calculation because the dimensionless character of the B/C ratio made it more applicable to the development of the interchange guidelines. On the basis of the results of the sensitivity analysis, it was determineu that the guidelines should include both major and minor traffic demands in the base year as independent variables. The base year is defined as the year the new interchange or intersection would be opened to traffic.

The analysis indicated that interchanges were more economically viable than signalized intersections as an alternative to two-way stop-controlled intersections on rural expressways. The signalized intersection's main benefit is a reduction in accidents; however, this benefit appears to be negated by the signal's higher operational costs whenever the minor road demand is less than one-half that of the major road. Moreover, even when this demand ratio is exceeded, the interchange consistently yields about $4 million more in benefits than the signalized intersection. This incremental benefit is generally sufficient to yield a B/C ratio of 2 to 3 over the signalized intersection under the full range of traffic demands. As a result, the interchange appears to be a more viable alternative than the signalized intersection on the basis of the economic and traffic conditions assumed for this study.

In recognition of the large degree of uncertainty in the estimate of the road user cost components, a range of B/C ratios was considered in the development of the guidelines. In particular, a range of major/minor road traffic demand combinations were evaluated using the computerized analysis methodology to find combinations that would yield B/C ratios of 1.0 and 2.0. Demand combinations that result in a B/C ratio of 2.0 imply that the road user benefits associated with a specific alternative design outweigh the incremental costs of this alternative by a factor of 2.0. Experience with the methodology suggests that this "factor of safety" of 2.0 should provide a traffic demand threshold that minimizes the uncertainty associated with the assumptions made for the analysis. Traffic demand combinations that result in B/C ratios between 1.0 and 2.0 suggest that more detailed, site-specific examination is needed to determine whether special circumstances exist that are contrary to the assumptions made in the analysis.

Guidelines

The benefit-cost approach used to develop the guidelines focuses only on economic factors. The decision to implement a particular alternative should also consider its social and environmental implications. In this regard, the results of the economic analysis should be considered as only one component of a more comprehensive alternative evaluation process. The guidelines herein are directed toward rural expressways and are based on economic and traffic conditions in Nebraska. All of the aforementioned assumptions and unit prices (particularly those for accidents) should be reviewed before application in other areas.

The analysis considered three geometric scenarios for the junctions. The first assumes that the intersection and the interchange have four approach legs and a two-lane cross section on the minor road. This is the most common configuration for both junction types in rural areas. The traffic demand-based guideline for this interchange scenario is shown in Figure 3.

Two lines are shown in this figure, which divide the graph into three regions. The lower line represents the base year traffic demand combinations that result in a B/C ratio of 1.0. The upper line represents the demand conditions coinciding with a B/C ratio of 2.0. The region in which the combination of major and minor road traffic demands fall indicates the recommended action. If the combination falls above the upper line, the interchange would be economically justified in most situations. If the demand combination falls below the lower line, there is not sufficient traffic demand to economically justify the cost of the interchange.

If the demand combination falls between the upper and lower lines, a more detailed examination is needed to ascertain the economic need for an interchange. This examination should determine if the assumptions made in this analysis are representative of the particular location. If they are, the interchange may be economically justified. If they are not, a benefit-cost analysis using the site-specific conditions may be needed to determine whether an interchange is justified.

Examination of Figure 3 indicates that road user benefits are relatively insensitive to major road traffic demand. Minor road demands of 2,000 and 4,000 vpd appear to define minimum threshold values for junctions with major road demands ranging from 4,000 to 15,000 vpd. These findings are similar in trend and magnitude to those of Van Every (*4*), who recommended using minor road demands of about 3,000 and 4,000 vpd for B/C ratios of 1.0 and 2.0, respectively.

The second scenario assumes that the conventional diamond interchange will have four approach legs and a four-lane cross section on the minor road. The traffic demand-based guideline for this scenario is shown in Figure 4. The general trend of the B/C lines in Figure 4 is similar to those in Figure 3; however, the higher cost of the four-lane cross section has shifted the lines upward toward higher minor road

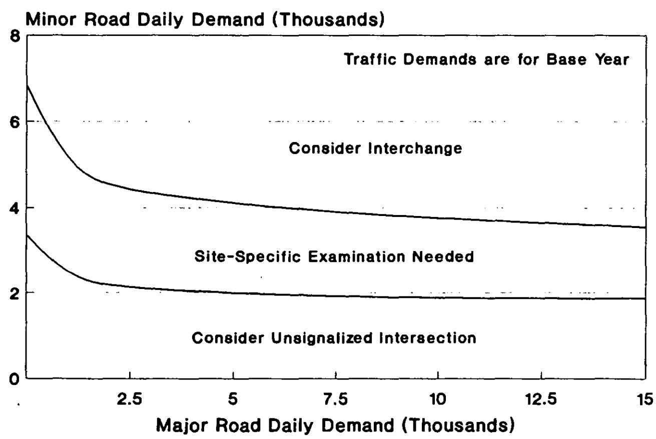

FIGURE 3 Interchange guidelines based on four approach legs and a two-lane minor road.

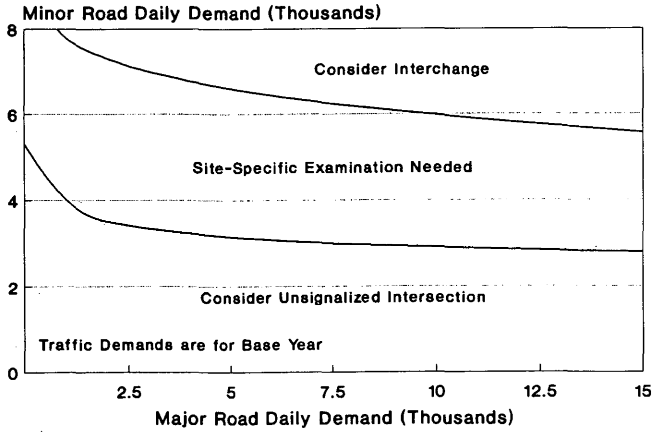

FIGURE 4 Interchange guidelines based on four approach legs and a four-lane minor road.

volumes. This shift stems from the need for more minor road demand (and related operating costs) to justify the higher construction cost of a four-lane bridge.

The third scenario assumes that the intersection and the interchange have three approach legs (i.e., a T-junction) and a two-lane cross section on the minor road. The traffic demand-based guideline for this scenario is shown in Figure 5. The general trend in the B/C lines in Figure 5 is also similar to those in Figure 3; however, the absence of a through movement on the minor road eliminates some of the user costs that are incurred at a four-leg intersection. As a result, more total minor road traffic (representing only turn movements) is needed to justify the cost of the interchange.

The analysis of a three-leg junction was conducted using the same methodology as that applied to the four-leg junction. The only difference was that one approach leg was specified as having zero demand. The equations for calculating the operating costs are valid for this application; however, the equations for predicting accidents become more of an approximation because they were derived from data for four-leg intersections. In spite of this limitation, the results appear to be generally consistent with experience and should be considered representative.

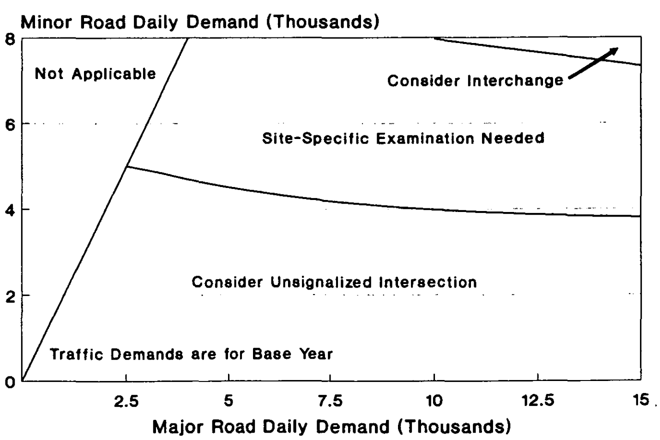

FIGURE 5 Interchange guidelines based on three approach legs and a two-lane minor road.

A fourth region, labeled "Not Applicable," is also shown in Figure 5. This region represents the traffic demand combinations that are not possible at T-junctions on the basis of the assumed 50/50 directional split in traffic demand on both intersecting roadways.

CONCLUSIONS

Examination of the benefit-cost analysis procedure indicated a strong sensitivity to minor road demand and discount rate. In general, road user benefits increased with increasing minor road demand and decreasing discount rate. Major road demand, traffic composition, and annual traffic growth rate did not have as strong an influence on user benefits.

The benefit-cost analysis of the signalized intersection and interchange indicated that the interchange was a more economically viable alternative than the signalized intersection. The signalized intersection's main benefit is a reduction in accidents; however, this benefit appears to be negated by the signal's higher operational costs whenever the minor road demand is less than one-half that of the major road.

Three geometric scenarios were formulated for the intersection and interchange. The first considered a four-leg junction with a two-lane minor road. The second considered a four-leg junction with a four-lane minor road. The third considered a three-leg junction with a two-lane minor road. Three figures were developed relating the major and minor road daily traffic demands that would economically justify an interchange in terms of benefit-cost ratios of 1.0 and 2.0. When the major road demand exceeds about 4,000 vpd, the minor road demands that provide a 2.0 benefit-cost ratio are about 4,000, 6,500, and 8,000 vpd for the three scenarios, respectively.

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