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16. Abstract

The focus of this project is to use a standard methodology to evaluate the Super Two concept for improving the geometrics of two-lane, two-way rural highways. This concept modifies existing lane usage by providing some form of alternating passing lanes on a reconfigured 44 foot cross section. The study includes the determination of signing and striping requirements. It also includes developing criteria for a test roadway and proposed geometric design requirements. Deliverables include a literature and benchmarking report, a feasibility study on the refined Super Two design, and proposed design criteria including drawings. A project summary report will contain the full description of all design alternatives with accompanying drawings to permit full implementation.

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Analysis of an Improved Two-Lane, Two-Way Highway Geometric Section (Texas Super Two).

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Implementation Statement

The results of this project will yield the documentation necessary to readily produce construction project plans for use in pilot projects in the Childress District. If the design proves successful, TxDOT highway design criteria would then be augmented with a mid-range alternative to providing a full four-lane highway. This option will prove to be particularly attractive in those areas where traffic volumes are marginal and right-of-way costs are high. The Super Two design would provide a solution for the problem of safe passing without the necessary sight distances. Additionally, as the design was proliferated across the State, driver behavior would be altered to operate on the roadway as it was designed. The best means to convey the findings of this research is through the project summary report and the design drawings.

Disclaimer

The contents of this report reflect the views of the authors, who are solely responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation.

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CHAPTER 1: BACKGROUND, IMPLEMENTATION, AND WORK PLAN STATUS

Background

The recent statutory change in the speed limit has had a dramatic impact on rural two-lane, two-way traffic. In years past, heavy trucks tended to utilize the interstate highway system because at the 55 mile per hour (mph) speed limit, it provided the least path of resistance. While actual distances driven were somewhat greater, trip times were roughly equal when compared to the use of rural highways. Some delays were due to driving through towns. The new speed limit changed that equation. By being able to travel at greater speeds, truck drivers can now save time by taking more direct routes to their ultimate destinations using the rural highways. Thus, the percentage of truck traffic on the rural highways has increased. With this increase in large vehicles comes an increase in the number of passing movements required by passenger vehicles. This condition is a direct function of the allowable speeds for the various types of vehicles. While passenger vehicles may travel at speeds up to 70 mph, trucks are restricted to 60 mph and school buses are restricted to 50 mph. This relative speed deviation creates a condition where drivers in open terrain, like that in West Texas, are tempted to execute potentially unsafe passing movements for three reasons. First, because the terrain is relatively flat drivers believe they have the necessary passing sight distance when often they do not. Secondly, because the traffic is generally light, drivers are more confident they can execute this movement safely. Third and most critical for this study's purpose, drivers know that they must execute a passing movement at some point if they do not want to follow a slow moving vehicle to the next town because there are no alternatives.

This condition is further exacerbated by a behavior characteristic of Texas drivers. It is customary in rural areas to ease onto the shoulder of the road and permit a faster moving vehicle to pass without having to pull completely into the oncoming lane of traffic. This practice has been carried to the point where a driver in the oncoming lane who sees a passing movement being executed ahead of him will also pull onto the shoulder, thus temporarily making a three-lane road out of a two-lane one. While this certainly speaks well for the courtesy of Texas drivers, this habit creates unsafe driving conditions and causes structural damage to the paved shoulders. In order to change this driving habit, two conditions must exist. First, the driver must be certain that an opportunity to pass a slower moving vehicle will occur in a reasonable amount of driving time. And secondly, the driver must also be certain that the safe passing opportunity will occur independently from the status of oncoming traffic.

The ideal solution to this problem is to widen these rural roads and provide a four-lane highway. However, between $1.0 and 2.0 million per mile of construction cost, this is not economically feasible (Operation and Procedure Manual, 1994). The logical alternative is to determine at what volumes the additions of periodic passing lanes are justified and develop several possible design alternatives to provide this capacity. Most states provide passing lanes on two-lane roads where passing sight distances are impossible to attain on hills and around short radius horizontal curves. Nevertheless, only a few have experimented with providing these lanes on terrain with adequate sight distances. New Mexico is one nearby state that has experimented with this concept. US Highway 64/87 between Raton and Clayton utilizes a Super Two style design (Eyler et al. 1996) with widened sections at five to eight mile intervals that permit passing. Thus, drivers of passenger vehicles know that a passing section will be available every two to five minutes.

Several foreign countries have also taken a similar interest in enhancing the safety of their highways without incurring the construction costs of building four-lane highways. Chapter 2 contains the details of Super Two design geometry currently used in Canada, Mexico, and Germany. The recent enactment of the North American Free Trade Agreement (NAFTA) makes the study of the Mexican and Canadian Super Two standard designs very important. At this point in the study, it appears that the Mexican section will be most easily adapted to use in Texas. Considering the relative proximity of Mexico and the relative percentage of Mexican traffic as opposed to Canadian traffic, modeling the Texas Super Two design after the Mexican approach makes more sense. The German design springs from a severe restriction on the amount of available right-of-way in European countries. Therefore, it seeks to minimize the required cross-section. It only requires a total of 12 meters from shoulder to shoulder. But to achieve this, the Germans utilize 0.25 meter shoulders and require a 0.5 meter separation between the opposing lanes with a physical barrier if possible. This is not typical of the cross-section generally presented to U.S. drivers on rural roads. This type of cross-section will not only confuse them, but the virtual lack of shoulders would present a hazard in a stopped vehicle situation. The Canadian design uses 1.2 meter shoulders on both sides of the road, and the Mexican design provides a wider shoulder on the opposing lane side of the passing lane side. The Mexican design makes more sense, so a breakdown on either side of the Super Two section will leave sufficient room for one unobstructed lane of traffic in each direction.

A Mexican study of truck lanes was published in the *Transportation Research Record* (Frost et al. 1995). This study indicated that benefits accrued from increased travel speeds as measured by the World Bank method outweighed the construction costs by as much as five times (Frost et al. 1995). Turkey has considered a similar plan as an option to leverage scarce construction dollars while enhancing safety as an interstate highway system is built. Studies have also been done in Great Britain (Technical Memorandum, 1996), China (Mendoza et al.1995), and Australia (Taylor et al. 1988) regarding the provision of alternating passing lanes to relieve congestion and enhance safety on two-lane roads with a high percentage of truck and bus traffic. The British study is particularly interesting because one does not consider Britain as a country with many long rural roads. The motivation in this study was to minimize right-of-way acquisition costs in a country where available right-of-way is extremely rare and comparatively costly. This parallels the current environment in Texas, especially when one considers Super Two as a means to increase highway capacity within existing right-of-way limits.

This project has three major phases. First, a bench marking analysis of the literature, current TxDOT design criteria and cross sections must be conducted. Also assembly of appropriate standards for geometric design, signing/striping, and minimum traffic volumes must be performed. The output of this initial effort will be a set of potential alternatives that can be evaluated in the next phase. It will also try to estimate traffic volume justification criteria (maximum and minimum ADT's) which will permit this design to be an alternative to four-lane highways. The second phase will entail a formal feasibility study to assess the costs and potential benefits of each of the various Super Two alternatives. The study will be open ended and evaluate all possible alternatives with an eye to reducing the field to the two or three best options on which detailed design and cost estimates can be assembled. Benefits and costs due to delay attributed to differential speed and headway will be estimated using queuing theory (Harwood, 1991). At the outset, a limited Monte Carlo simulation (Harwood, 1991) was considered as a way to more accurately estimate delay savings benefits, but this was determined to be unnecessary as the ADT's involved will not justify themselves on a classically derived warrant basis. The final phase will be to formalize the results of the first two phases by producing the necessary design documents to bring the concept to life on a test project. The documents will include engineering drawings, signing and striping plans, as well as a draft implementation plan, which will include proposed traffic volume criteria, cost estimate information, and a formal constructability review for each design alternative.

Implementation

The results of this project will yield the documentation necessary to readily produce construction project plans for use in pilot projects in the Childress District. If the design is successful, TxDOT highway design criteria would then be augmented with a mid-range alternative to providing a full four-lane highway. This option will prove to be particularly attractive in those areas where traffic volumes are marginal and right-of-way costs are high. The Super Two design would provide a solution for the problem of safe passing without the necessary sight distances. Additionally, as the design is proliferated across the State, driver behavior would be altered to operate on the roadway as it is designed. The best means to convey the findings of this research is through the project summary report and the design drawings.

Work Plan Status

The Work Plan contains six tasks that are directly related to the objectives detailed in the research problem statement. The plan follows a ordered sequence, and coordination with the Program Coordinator and Project Director will be maintained throughout the process. Each task is oriented on efficiently and effectively producing the project deliverables. The plan is summarized in the Project Abstract shown in the previous section. The details of the Work Plan are as follows.

Task 1: *Literature Review*
This task is complete. A comprehensive review of the literature on this subject was completed to provide a solid theoretical and anecdotal foundation for the development of Super Two design alternatives. Particular attention was paid to work done by other public agencies, both foreign and domestic, in the design of Super Two cross-sections as well as ancillary features such as signing and striping. The calculation of construction and maintenance project costs and benefits was also closely monitored. Specifically, other state departments of transportation, the Federal Highway Administration, and the transportation ministries of Mexico, Canada, Germany, Turkey, Great Britain, and Australia were contacted to obtain examples of two-lane, two-way passing lane design criteria and specifications. Additionally, the subject of engineering economics was thoroughly researched to ensure that the latest developments in Life Cycle Benefit-Cost analysis for transportation projects are found for use in the subsequent tasks.

The primary focus of the literature review was to identify different passing lane design alternatives that might be adapted to the Super Two concept. Specific geometric design criteria were sought. The search included the following subjects:

- Passing lane design criteria.
- Average Daily Traffic range for passing lanes.
- Accident rates and costs due to lack of passing lanes.
- Signing requirements.
- Striping requirements.
- Cost factors.
- Computing benefits accrued to reduced delay.
- Highway life cycle cost computations.
- Applications of queuing theory to computing expected delay due to relative speed differential.
- Human factors regarding minimum acceptable delay in passing.

These areas of study provide a framework on which the comprehensive feasibility analysis will be performed. Chapter 2 contains the details of the literature review.

Task 2: *Develop Super Two Design Alternatives*
This task is essentially complete. Based on the results of Task 1, a comprehensive set of Super Two design alternatives was assembled. Each alternative was developed to the level of design detail necessary to permit accurate cost estimates to be made on its initial and life cycle costs. A specific case study of traffic data and geometry (US 87 from Dalhart to the New Mexico State Line at Texline) was applied to each potential design in an effort to determine various alternatives for detailed design in Task 3. This case study was selected because US 87 continues into New Mexico and between Clayton and Raton contains a series of Super Two sections. A field study will be conducted in June which will sample traffic in Texas with no Super Two sections and in New Mexico between Super Two sections to provide field verification of the analytical analysis.

It was found that there are four basic types of Super Two designs. To standardize the various design approaches, researchers have set up the following classification system (details are shown in Appendix B, Super Two Design Alternatives):

- **Type A** (*Interior Passing)*: Passing lane is shifted to the interior of the section with alternating direction and center line for the length of the Super Two section.
- **Type B** (*Separated Passing)*: Passing lane is shifted to the outside of the section with a constant center line alignment and alternating passing sections which are linearly separated by some distance for the length of the Super Two section.
- **Type C** (*Overlapping Passing)*: Passing lane is shifted to the outside of the section with a constant center line alignment and alternating passing sections which are overlapped by some distance for the length of the Super Two section or a short stretch of four lane.
- **Type D** *(Isolated Passing)*: A single passing lane in one direction shifted to the outside of the section

Types B and C contain several variations that are detailed in the appendix. Each type has its advantages and disadvantages. Type A appears to be the best section for a highway which is emerging from a population center. It allows smaller vehicles to accelerate to design speed without delays from slow moving vehicles. Types B and C would be most useful in rural areas where ADT might increase after a junction with another rural road. Additionally, these alternatives can be "mixed and matched" based on right-of-way constraints and limitations placed on the traffic by horizontal and vertical alignment. Type D provides a means to enhance safety at a specific point where a high percentage of slow moving vehicles are present due to some specific traffic generator like a cotton gin, a grain elevator, or a factory. At this point in the study, it appears to be wise to include all alternatives in the subsequent analysis and furnish the Department with a robust set of means to satisfy a varied set of design problems.

The queuing model is being used to calculate the expected savings in delay times for differential speed and headway for each alternative. Actual traffic data from the case study is being used to test the queuing model. The fundamental approach is fairly straightforward. The average number of cars in a queue behind a truck is a function of the difference in vehicular speed limits, the ADT, and the physics of vehicle acceleration and maneuvering. The average queue length can be estimated during peak hour traffic as defined by AASHTO. Assuming no opposing traffic and sufficient passing sight distance, the first car in the queue will be delayed by the time it takes to accelerate from 60 to 70 miles an hour, pass the truck, and pull back into its lane. The second car will be delayed for the period that it takes the first car to pass plus the period it takes the second car to pass and so forth until the queue is empty. This is a single channel queuing model. When a queue arrives at a Super Two section, all the cars can pull into the passing lane and accelerate simultaneously. Therefore, the delay will be equal to the passing time for one car plus the time it takes for the total number of cars to pass at the design headway. This is a multi-channel queuing model. The optimum Super Two section will need to be long enough to empty the average queue and the distance between Super Two sections should be directly related to the time it takes for the average queue to build. Thus, queues are built between Super Two sections and emptied inside the Super Two sections. Appendix B contains details of this analysis and an example based on the Dalhart case study traffic characteristics.
This information will be used along with the conceptual cost estimates to optimize the distance between passing zones for those alternatives where it is appropriate. Finally, the results of this analysis will be used to identify those alternatives that promise to present the best solution to this problem.

Task 3: *Detailed Design of Best Alternatives*
This task is about 40% complete. The research assistants have been trained in the use of the MICROSTATION CAD program, and sample files of standard TxDOT plans have been procured from the Childress District. One example of each Super Two Section Type will be designed in detail. This will include cross-sectional geometrics, signing, striping, barriers, flush median widths, transition configurations, and lengths of designated passing and no-passing dedicated lanes. These designs will be drawn up to a level of detail that will permit an accurate quantity take-off to be conducted. Traffic volume criteria will be associated with each alternative, and savings in vehicle delay times will be estimated to permit a benefit determination using the World Bank method for each alternative. A detailed cost estimate will be completed

from the quantity take-off, and this will serve as the estimated cost of construction. The estimate will be verified by using FPS-19 software to calculate lifecycle cost information.

The details of design will be developed using MICROSTATION CAD software. A draft of these details will be submitted to the Project Director for comments and approval. The details will be based on standard TxDOT details and drawn in a form that will permit direct transfer to construction documents once the designs are approved. Any comments received from the Project Director will be incorporated before the final cost estimate is finished to ensure the accuracy of the estimate.

Task 4: *Feasibility Study*
This task is underway and about 25% complete. All basic cost data has been gathered. The World Bank methodology is available and verified to be appropriate for this application. FPS-19 is on-hand, and initial development of the economic algorithm is underway. The cost and expected FPS-19 design life estimates from Task 3 will be used as the primary basis on which to conduct the work associated with this task. The feasibility study will contain three major items of work. First, the development of an appropriate economic unit with which to compare the various alternatives with current alternatives will be done. The researchers will use this to develop a Cost Index Number (CIN) based in cost index theory for each of the alternatives. It is anticipated that a unit such as cost per ESAL-year per square meter of paved surface will be used. This particular unit allows not only the typical construction costs to be considered but also the function of economic efficiency due to demand. The function of economic efficiency gives a more global view of the relationships between each alternative. It also will account for those roads with low ADT, but a high percentage of trucks. In fact, this approach will also permit an evaluation of the range of feasible ADT's that must be in operation to justify Super Two over a two-lane, two-way or four-lane divided highways.

The second item of work will be to develop a Benefit Index Number (BIN) from the information gleaned from the queuing theory analysis. The BIN will then allow the researcher to conduct a classic Benefit-Cost ratio analysis using the following test:

BIN/CIN $\geq$ 1.0 to create a viable alternative.

This analysis will permit the rank ordering of each alternative and further reinforce the evaluation of the recommended Super Two ADT range. For instance, if one alternate Super Two design is found to have a B-C ratio which falls between the B-C ratio of the current two-lane design and the current four-lane design, we will have proven that Super Two is a promising alternative for those areas where actual and forecast traffic volume rides the ADT break point between the two current designs. Additionally, this information can be used to validate certain design criteria and optimize the design parameters of each alternative with respect to costs and benefits.

The final portion of the feasibility study will address geometric concerns with respect to right-of-way. It is reasonable to assume that Super Two could be used as an interim solution for expanding a two-lane stretch of highway to four-lane capacity in an area where traffic is growing at a slow but steady rate. Thus, the model for phased acquisition of right-of-way and phased increase in highway capacity is appropriate. The study will look at the costs and benefits associated with planned increases in highway capacity and make recommendations on the following items:

- Timing between capacity upgrades.
- Optimal ROW acquisition strategy.
- Trigger levels of traffic volume preceding highway capacity upgrades.
- Expected economic justification data.

Task 5: *Prepare Final Design Criteria for Implementation of Test Roadway*
This task is scheduled to begin in May, 1998. The results of the previous tasks will be synthesized in this task. Design criteria for roadway geometrics, signing, striping, alignment, and other pertinent aspects will be formalized. The output of this task will be a set of engineering plans and drawings using MICROSTATION CAD software that can be directly converted to project construction plans. It will also include such contract general and special provisions as may be required to successfully implement this initiative as well as any recommended changes and additions to current TxDOT specifications.

Task 6: *Prepare Project Reports*
Three major reports will be prepared for this project. The first will be a research report which outlines the findings of Tasks 1and 2. The report will focus on the results of the literature and provide a benchmark on the international level of the status of Super Two Type designs. It will include a concept level design for each potential alternative along with cost and benefit data that has been developed at the time of publication. The second report will be the feasibility evaluation report called for in the project problem statement. This report will be a summary of Tasks 3 and 4. The third report will be the project summary report that provides a complete summary of the project results along with detailed recommendations for implementation.

CHAPTER 2: LITERATURE REVIEW

Super Two refers to a non-freeway or controlled access at-grade roadway with a single through lane per direction. The idea behind Super Two is to increase the capacity and safety of the existing two-lane two way highways.

The key elements of a Super Two Highway (Eyler and Poletz, 1996):

1) Full width lanes, paved shoulders and clear zones
2) A designated passing lane
3) Limited access, with turn lanes for all permitted turns
4) Horizontal and vertical curves with high speeds
5) Passing lanes, speed differential, and truck lanes
6) Provisions for expansion to freeway or divided roadway
7) Proper interchange or intersection design for a two-lane freeway

For a new facility to be classed Super Two, most guidelines should be met. For upgrading an existing roadway, these defining features can serve as a menu of improvements for consideration.

Design Philosophy

The philosophy of Super Two is to provide smooth traffic movement and overtaking maneuvers in the traditional two-lane two-way highways. If constructed as a two-lane road the Super Two will provide many of the facilities of a four-lane highway. In planning a regional road system, the Super Two would be facility typically used for minor and low-volume principal arterials.

Design Features

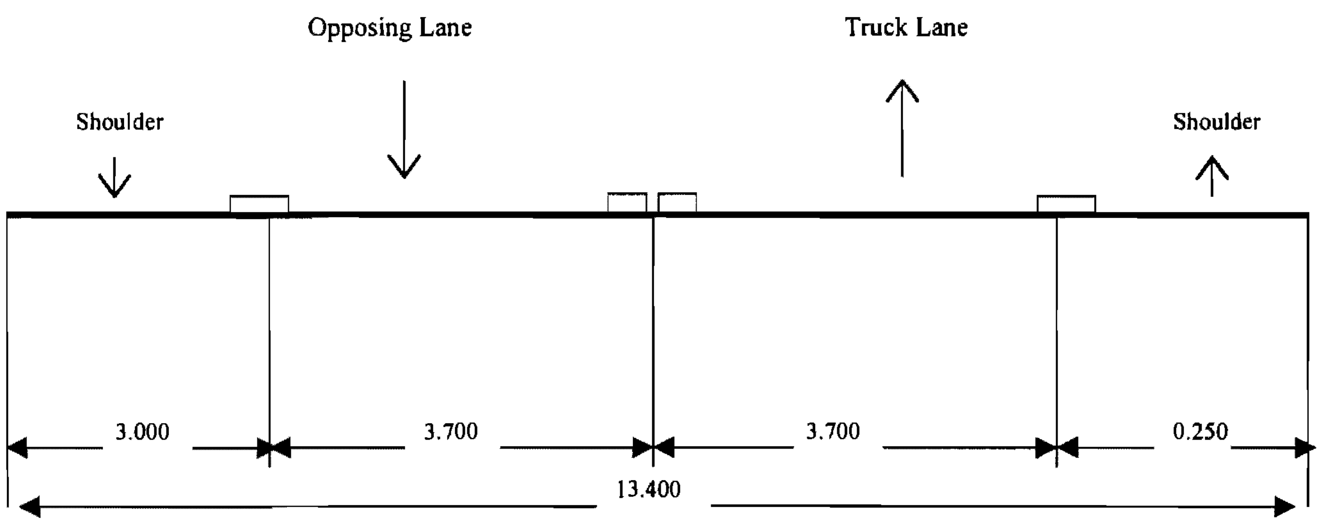


Figure 2-1: Current Two-Lane Rural Highway Cross-section

The typical two-lane highways have two middle lanes 3.2-3.7 m in each direction (as shown in the previous figure) and shoulders 0-3.0 m each. That makes the total width of the highway 13.40 m. The cross slope is 2%. There is no passing lane in it, and shoulders are design fully surfaced.

Design Speed and Average Daily Traffic

The Design Speed should range from 80 to 110 km/h when an existing two-lane highway is upgraded to Super Two. In all new construction and reconstruction the Design Speed of 100 to 110 km/h should be used (Minnesota, 1996). In all cases, when upgrading the existing roadway, the designer should apply the speed that is greater than or at least equal to the posted speed (Technical Memorandum, 1996).

The ADT value of 2000 is considered to be the critical ADT between two and four lane highways. In a study published in the Transportation Research Record (TRR) 1303, "Warrants for Passing Lanes" (Taylor et al. 1988), shows that Passing Lanes on rural two-lane highways have favorable benefit/cost ratio at AADT's of 6500 and greater. The length of a passing lane is dependent on the volume of vehicles per hour (vph) for the project. The optimal length of passing lanes to reduce platooning is 0.8 to 1.6 km (Technical Memorandum, 1996). General guidelines for the development of design length is as follows:

VPH One Way	Length of Passing Lane
400	1.2 to 1.6 km
700	1.6 to 2.0 km

The spacing design for passing lane is dependent on traffic volume. For a vph of less than 700 this spacing may vary from 16 to 24 km. On the other hand this spacing may vary from 5 to 8 km or more for a vph of 700 or more (Technical Memorandum, 1996).

Typical Three Lane Sections

The three-lane section consists of an added lane in the middle to provide passing facilities in alternate directions. This change of direction of the middle lane may be uniform or traffic actuated, depending on situation. The center lane added to the two-lane section is usually of the same width. The shoulders of a three-lane section may be narrower than that in the two-lane section, as being practiced in Germany, Canada, Mexico, and Turkey. Brief descriptions of the German, Canadian and the Mexican cross sections are as follows:

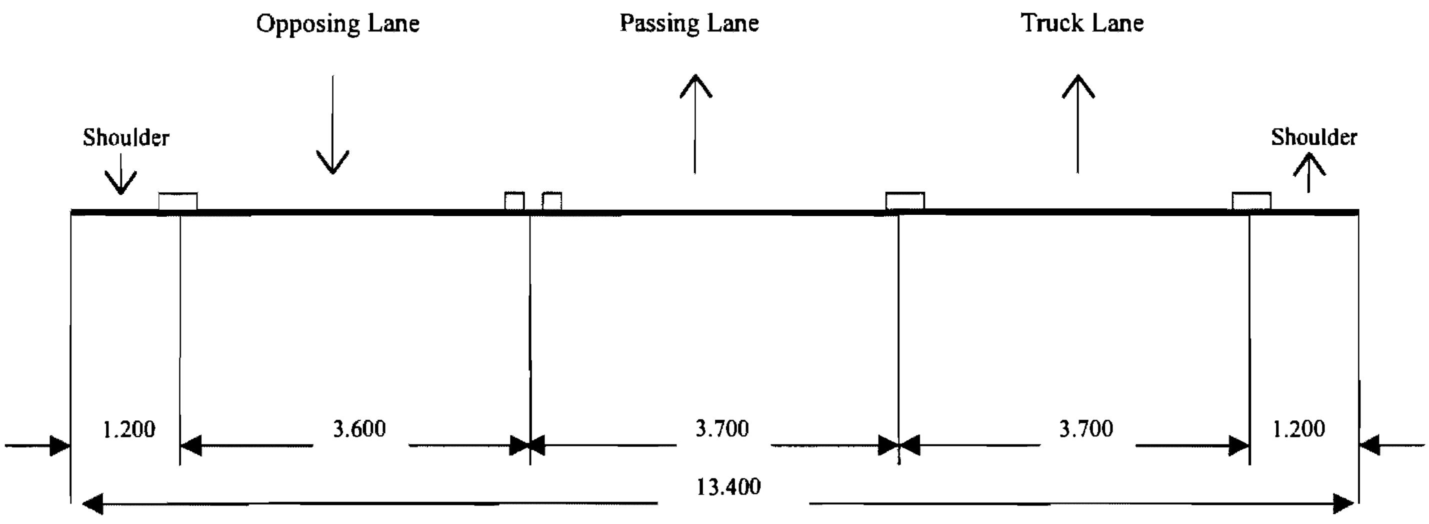


Figure 2-2: Three Lane Highway Cross Section in Canada

The design consists of two 3.7 m lanes and a 3.6 m lane with shoulders of 1.2 m each. The direction of passing will have a 3.7 m and a 3.6 m lane as shown in the above figure. The total width of the highway remains 13.40 m (Frost et al. 1995).

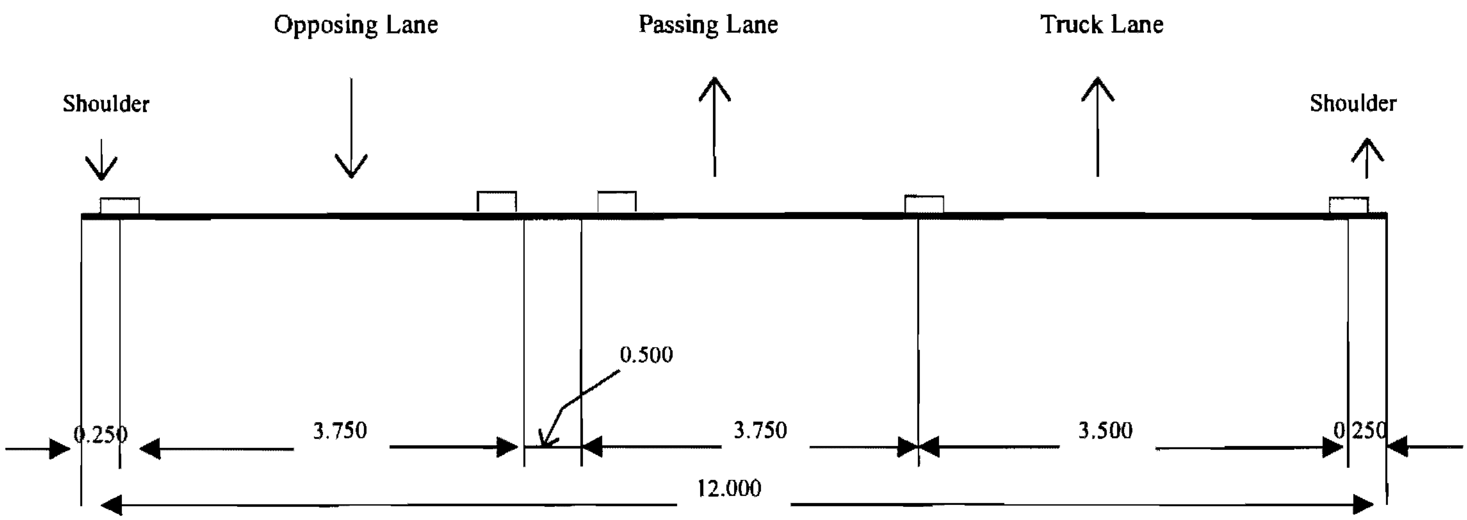


Figure 2-3: Three Lane Highway Cross Section in Germany

In Germany, the highway is designed with two 3.75 m and one 3.5 m lanes. The middle lane with a width of 3.75 m is provided to facilitate passing in alternate directions shown in Figure 2-3. The shoulders are of 0.25 m each making the width of the highway 12.0 m. This design is suitable for an AADT less than 2000. Overtaking is prohibited in the opposing lane (Frost et al. 1995).

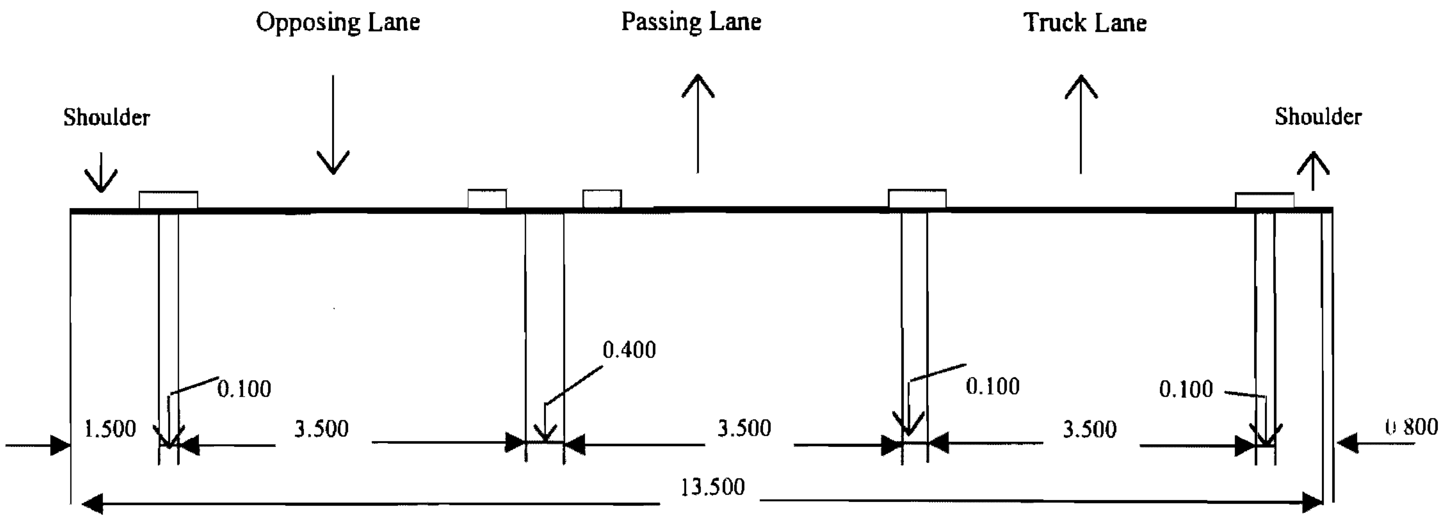


Figure 2-4: Three Lane Highway Cross Section in Mexico

This design alternative practiced in Mexico consists of three equal lanes of 3.5 m each with a central median of 0.4 m. The shoulder in the direction of passing is limited to 0.8 m and that on the reverse direction is 1.5 m as shown in figure 2-4. This change in shoulder width may provide some added safety to the driver (Mendoza et al. 1995).

Benefit/Cost Analysis of Super Two Highways

The growing need for the two-lane highways with more capacity, mobility and safety has lead researchers to put emphasis on alternative designs. On the other hand, limited funding and environmental issues are big concerns in opening new corridors to handle heavy traffic flows. As a result, the Super Two concept has recently started drawing more attention. There are more than 3 million miles of rural highways in the United States; this represents about 97 percent of the total rural system and 80 percent of all U.S. roadways. It is estimated that about 68 percent of the rural transportation and 30 percent of all travel in U.S. is done on the rural two-lane system.

Benefits of a passing lane include reductions in delays and accidents. In order to evaluate the effectiveness of passing lanes, the cost savings of the motorists over a wide range of traffic volumes should be compared to the construction and maintenance cost of the passing lanes. The reduction in delays provided by a passing lane results in operational cost savings to the road users. A unit value of time that is usually expressed in dollars per traveler hour is multiplied by the amount of time saved in order to compute the time cost savings. Besides the needs for updating these values to current price levels, travel time value is sensitive to trip purpose, travelers' income levels, and the amount of time savings per trip. According to AASHTO, the time savings is divided into three categories and can be expressed as the type of trip and as a function of time savings per trip (Taylor et al. 1988).

The three categories time savings can be divided into are listed below.

1. Low time savings (0-5 min): For work trips and average trips, the values of time per traveler hour are suggested as $0.48 (6.4 percent of average hourly family income) and $0.21(2.8 percent of average hourly income).
2. Medium time savings (5-15 min): For work trips and average trips, the values of time per traveler hour are suggested as $2.40 (32.2 percent of average hourly family income) and $1.80 (24.4 percent of average hourly family income).
3. High time savings (over 15 minutes): For work trips and average trips, the value of time per traveler hour is suggested as $3.90 (52.3 percent of average hourly income).

Below is an example of field study conducted in Michigan's two-lane rural highways. The delay benefits were calculated using the values of travel time for both average trips and work trips. According to the 1980 Census data, the average annual family income is $27,000. That gives the average hourly family income as $13.00, considering 2,080 working hours in a year. For average trips the value of travel time per hour per traveler is taken as $0.36, which is 2.8 percent of the average hourly income of $13.00. For work trips, this value is taken as $0.88 (Taylor et al. 1988).

Table 2-1: Cost Benefit Due to Passing Lanes for Typical Cases (Taylor et al. 1988)

Volume- Both Directions Veh/hr	ADT	Delay Benefit (sec/Veh)	Delay Benefits for Average Trips		Delay Benefits for Work Trips	
			$/hr	$/year	$/hr	$/year
WITH TWO PASSING LANES-ONE IN EACH DIRECTION (See Figure B-2) (Type B Design with Separated Passing Lanes)						
500	5000	28.88	2.2	8030	5.1	18615
800	8000	32.76	4.0	14060	9.2	33580
1000	10000	37.84	5.8	21170	13.4	48910
WITH ONE PASSING LANE IN DIRECTION 1 (See Figure B-4) (Type D Design with Isolated Passing Lanes)						
50	5000	17.29	1.3	4745	3.0	10950
800	8000	17.56	2.2	8030	5.1	18615
1000	10000	17.91	2.8	10220	6.5	23725
WITH ONE PASSING LANE IN DIRECTION 2 (See Figure B-4) (Type D Design with Isolated Passing Lanes)						
500	5000	14.38	1.1	4015	2.5	9125
800	8000	19.06	2.3	8395	5.3	19345
1000	10000	23.40	3.6	13140	8.3	30295

Accident Cost Savings

An analysis of accidents on two-lane highways with and without passing lanes determined the effectiveness of a passing lane in reducing the number of accidents. The accident data is obtained from the state file for all two-lane road sections on rural highways throughout Michigan for 5 years, 1983 to 1987. The accident rates classified by severity for various ADT ranges is given below (Taylor et al. 1988).

Table 2-2: Accident Rates By Severity On Two-Lane Rural Highways In Michigan

	Without Passing Lane				With Passing Lane			
Year	Injury Acc. Rate	Fatal Acc. Rate	P.D.O. Acc. Rate	Total Acc. Rate	Injury Acc. Rate	Fatal Acc. Rate	P.D.O. Acc. Rate	Total Acc Rate
FOR ADT 1-5000								
1983	61.0	2.2	203.1	266.3	49.1	0.0	183.8	232.9
1984	61.8	2.3	221.9	286.0	59.4	0.0	172.4	231.9
1985	60.4	2.3	242.6	305.3	40.6	0.0	283.0	323.7
1986	60.0	2.5	255.6	318.1	46.3	3.0	206.8	256.0
1987	59.5	2.5	259.5	321.5	14.6	0.0	249.4	264.1

	Without Passing Lane				With Passing Lane			
Year	Injury Acc. Rate	Fatal Acc. Rate	P.D.O. Acc. Rate	Total Acc. Rate	Injury Acc. Rate	Fatal Acc. Rate	P.D.O. Acc. Rate	Total Acc Rate
FOR ADT 5001-10000								
1983	72.5	2.1	169.1	243.7	63.8	2.5	181.2	246.3
1984	75.7	2.6	172.9	251.2	49.3	0.0	170.5	192.5
1985	79.0	2.6	210.0	291.6	80.9	0.0	183.9	263.3
1986	72.1	2.9	210.1	285.0	45.8	0.0	228.2	272.0
1987	73.1	2.6	204.5	280.2	59.1	0.0	168.6	260.0
FOR ADT 10001-15000								
1983	103.8	1.5	199.6	304.9	27.5	10.5	203.8	241.5
1984	109.4	2.8	217.5	329.7	39.8	0.0	183.0	222.8
1985	97.7	3.0	228.9	329.6	70.5	0.0	157.5	228.0
1986	99.9	2.2	245.4	347.5	63.3	0.0	328.0	391.3
1987	98.9	3.0	223.0	324.9	93.0	0.0	216.5	309.5

To better evaluate the effectiveness of the passing lanes in terms of accidents, the reduction of accident rates is given in the table below. The rates are classified by severity and for various ADT ranges.

Table 2-3: Average Accident Benefit Due To Passing Lane (Taylor et al. 1988)

	Fatal Acc. Rate	Person Killed Rate	Injury Acc. Rate	Person Injured Rate	P.D.O Acc.	Total Acc.
ADT < 5000						
Without Pl	2.4	2.9	60.5	96.6	236.5	299.4
Within Pl	0.6	0.6	42.0	62.8	219.1	261.7
Benefit	1.8	2.3	18.5	33.8	17.4	37.7
5000 < ADT < 10000						
Without Pl	2.6	3.1	74.5	123.3	193.3	270.4
Within Pl	0.5	0.5	59.8	94.1	186.5	246.8
Benefit	2.1	2.6	14.7	29.2	6.8	23.6
ADT > 10000						
Without Pl	2.5	3.0	101.9	168.7	222.8	327.2
Within Pl	2.1	2.1	58.8	94.6	217.8	278.7
Benefit	0.4	0.9	43.1	74.1	5.0	48.5

Accident Costs

The accident cost is calculated by using methods previously developed in a study for FHWA. This method gives direct, indirect, and total costs per fatal, injury, and property-damage-only (P.D.O) accidents in rural and urban area. The results are summarized below.

Table 2-4: Accident Costs by Severity (Taylor et al. 1988)

Area and Type of Cost	Fatal($)	Injury($)	P.D.O.($)	Average($)
Rural				
Direct	50654	9542	1600	5424
Indirect	1183580	5731	282	21356
Total	1234234	15273	1882	26780
Urban				
Direct	44071	8403	1872	3768
Indirect	1111355	4172	330	6364
Total	1155426	12575	2202	10132

Accident Cost Savings Analysis

The accident cost savings provided by passing lanes are computed with the following equation (Taylor et. al, 1980):

$$ACS=(AC)(365)(ARF)(ADT)10^{-8}$$

ACS = Annual accident cost savings provided by a 1 mile passing lane ($/year/mile)
AC = Average cost of accidents by severity
ARF = Average reduction in accidents by severity for different ADT values (per 100 million vehicle-miles)

The equation above is used to compute the safety benefits of a passing lane on rural two-lane highways in Michigan. The values of the average cost of an accident are taken as the total rural accident cost for fatal, injury, and P.D.O accidents. Direct costs are considered in the equation. Total benefits of the road users are calculated by adding the delay benefits to accident cost savings. The equivalent uniform annual cost (EUAC) is calculated based on a passing lane 1.0 mile long. The life of the road is taken as n=15 years. For i=5 and 10, the values of the capital recovery factor are calculated as 0.0964 and 0.1315, respectively.

The values of total benefits for average trips and EUAC for 5 and 10 percent discount rates are plotted in the figure below.

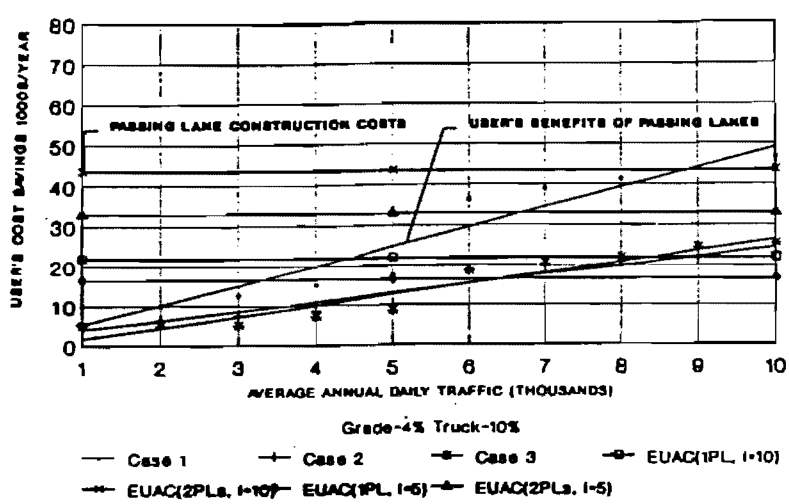


Figure 2-5: Comparison of cost and benefits for 4 percent grade, 10 percent trucks, and average trips on a typical road profile (Taylor et al. 1988).

As shown in Figure 2-5 all the warrants for a passing lane are met. But the user benefits are greater than the construction costs for a passing lane for all ADT values greater than 6500 for a discount rate of 5 percent. Similarly, for the same values of truck percentage, grade, and trip type the benefits are greater than construction costs for two passing lanes for ADT values greater than 9000 for a 10 percent discount rate.

CHAPTER 3: HUMAN FACTORS IN AUTOMOBILE DRIVING

The automobile has had enormous impact on life in the twentieth century, and it is reflected in automobile jargon and themes of motility becoming enmeshed in language depicting our emotional states and life events. Impulses are described as 'drives' in psychiatry, and a highly energetic person may be described as 'driven'. Similarly, an intense or active person may be called "accelerated", "a fast mover", "a big wheel", "a self-starter", or described as "in the driver's seat". Advancements in life seem to coincide unconsciously with forward locomotion while failures are seen as the equivalents of immobility.

Driving on a rural two-lane highway is a different experience for the driver than driving in an urban setting. It may sometimes cause boredom to the driver. First, there is no passing lane to overtake the slower moving vehicle, and frustration may occur when a driver finds there is no means of passing in the next few miles unless he or she overtakes the slower vehicle by crossing the center line. This may cause the driver to take an unwanted risk.

Automobile operation is governed by factors affecting the emotional and physical condition of driver. Ego-gratification, age, risk-taking behavior, time pressure, and inattention are some factors that may affect a driver's operation of a vehicle. Based on one research (Jackson et al. 1976) the gender of the driver and accompanying passenger(s) may also have effects on driving.

Driving and Speed

Waiting behind a slow moving vehicle for a relatively long period of time may break the patience of the driver and allure him to take the risk of meeting an oncoming vehicle with additional speed by crossing the center line. This kind of behavior depends on certain psychological factors and states of emotion of the driver in a given situation. In an attempt to find why drivers speed Gabany, Plummer and Grigg (1997) have found factors that affects speeding are (a) Ego-gratification, (b) Risk-taking, (c) Time pressure, (d) Disdain of driving, and (e) Inattention. According to their research, male drivers agreed more strongly than female drivers did with ego-gratification. Younger people agreed strongly with risk-taking and less strongly with time pressure. And females agreed more strongly than males with time pressures.

Ego-gratification

Drivers in a rural two-lane highway may want to overtake a slow moving vehicle with speeding just to satisfy his ego. Some factors concerning ego-gratification that cause drivers to speed are listed below.

- To scare their passengers
- To get a thrill out of breaking the law
- To get a thrill out of flirting with death and disaster
- To see how fast they can go before they get a ticket
- To defy the law
- To show off and attract attention
- To impress their passenger
- To see if they can get away with speeding

- Irresponsibility, immaturity and rebellion
- Because their friends speed
- To race each other
- To take risk
- Broken speedometers

Risk-taking

Drivers on a two-lane highway may take the risk of overtaking the slower moving vehicle by using the lane for oncoming traffic. This kind of risk taking behavior for speeding is governed by certain facts listed below.

- The fun of driving fast
- The feeling of drive fast
- The excitement rush from driving fast
- A sense of power and control
- A dislike of being passed by other drivers

A field study of risk-taking behavior of automobile drivers was done by Jackson and Gray (1976). They measured the time spent and the time interval before the next oncoming automobile while turning from a busy street without a stoplight. The results indicated that male drivers with same gender passengers had shorter wait times and drivers with mixed gender passengers had longer interval times as shown in the figure below.

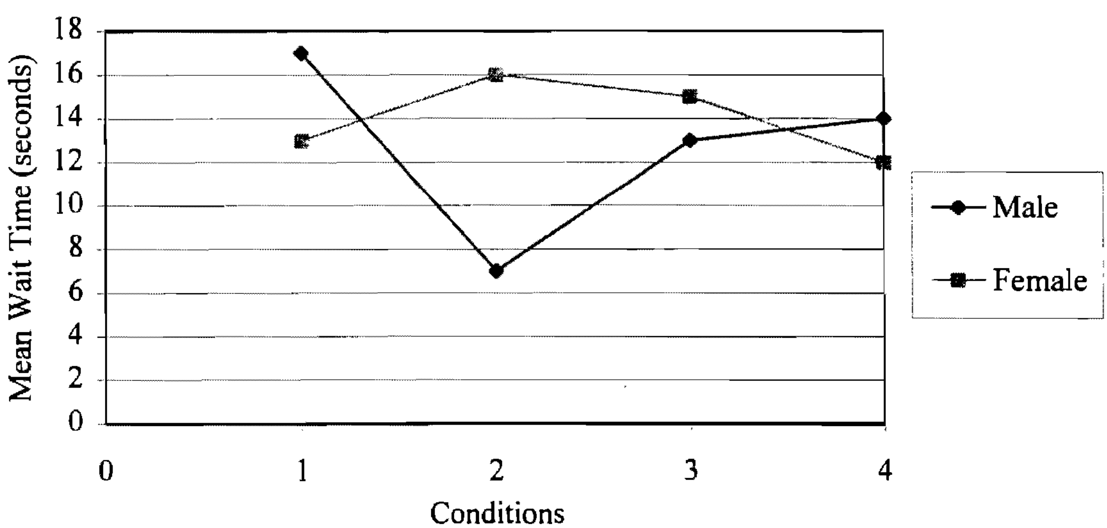


Figure 3-1: Male v. Female Wait at Intersection

The four conditions indicate the different combinations of genders of drivers and passengers.

The conditions are:

- Alone
- Same Gender
- Mixed Gender
- Opposite Gender

Figure 3-1 indicates that male drivers with same gender passengers have significantly shorter wait times than when they are alone or with female passengers. Adeptness in maneuvering an automobile might be considered a skill associated with masculinity. In order to appear successful, a male driver with a male passenger may be more willing to take an implied risk in this situation.

Another interesting finding is that drivers with passengers of both genders had significantly longer wait times. This may indicate that family groups were represented. This created a situation where risk-taking behavior could be more costly to the driver, i.e., a wreck could possibly injure loved ones. Risk-taking behavior may also be guided by the time pressure factor.

Time Pressure

Time pressure is always a factor affecting drivers who speed. Waiting at the back of a long queue can create extra pressures of feeling late. If someone has time restraints and finds himself at back of a queue, he will obviously try to change his situation. Most drivers speed because they are late, angry, or in a hurry. Some situations that cause drivers to feel the need to speed are listed below.

- An emergency
- Trying to get somewhere before it closes
- Trying to get home quickly
- Schedules include more things to do than time to do them
- Excited about getting somewhere.

Young men with limited driving experience or immaturity do not seem to perceive as many pressures of time. Older people with more responsibilities will agree with the time pressure factors than young people. Females agreed more strongly than males with the items underlying time pressures. It is generally accepted that females mature at a more rapid pace than males, and their perceptions of time pressures rather than ego-gratification may be more reflective in their driving speed.

Disdain of driving

This feeling may occur at the end of a long trip when drivers are close to there home or when driving becomes an unwanted job to the driver. Factors that affect disdain of driving are listed below.

- Every minute counts
- Time is money
- Fatigue and desire to get home quickly
- Driving becomes automatic
- Impatience

Inattention

Inattention may also cause speeding violations. Primary factors are:

- Not realizing speed
- Not paying attention to their speed
- Speeding unintentionally
- Distraction

It is clear that waiting in a queue on a rural two-lane highway may not be a pleasurable experience. The Super Two may be a justifiable solution to overcome this kind of inconvenience.

Driving Behavior

Although it has been considered that most accidents are caused by 'accident-prone' drivers, Forbes (1939) thought of the importance of ordinary drivers. Accident-prone drivers are only 1.3% of the driving population, and they are involved in 3.7% of accidents that occur. The remaining 96.3% of the accidents are caused by ordinary drivers. Only 10 to 15% of that 96.3% may be attributed to mechanical or environmental factors. The other 85 to 90% are believed to be caused by human failure (Lewin, 1982). In these situations the vehicles primary purpose of transportation may be neglected, and the driver uses his car as an instrument of self-aggrandizement (Grumet, 1989). A typical illustration of such a behavior is a farmer who works all day at his ranch exhausted by the heat and hard labor. At the end of the day, all this farmer wants to do is go home. Entering a powerful automobile, he experiences a cathartic release as he tries to reach home swiftly. On a rural two-lane highway, he may find a big truck moving at a speed of 60 mph or less, and his speed limit is 70mph. The technical operations of driving may be corrupted by an emotional need to unleash, upon the highway, feelings of escape, release, and freedom. His psyche is thus nurtured, his ego sustained, and the failings of the day undone.

Queue Psychology

There are two basic patterns of driving observed: individual and collective. A driver usually functions independently until he comes within about 200 feet of the vehicles in front of him and begins to interact within them (Herman et al. 1963). These interactions can be examined at the levels of traffic dynamics and psychodynamics. This kind of vehicular interaction may be observed when cars are moving on a busy, single lane of a rural two-lane highway. In a queue, all of the vehicles will follow the typical rules of a queue, which are move when the front vehicle moves and stop when the front vehicle stops. If one driver overreacts, for example slows down too vigorously, the drivers behind him will promptly follow suit. The disturbance will propagate back along the line like a " shock wave" or "pulse" that may eventuate in a stoppage of traffic or a chain of rear-end collisions at some distance behind. Drivers who are excessively sensitive may amplify the deceleration pulse while the relaxed driver will react in a smoother manner and slow down or stop much of the disturbance. The need to leave at least a two-second space between vehicles is underscored by the danger of collision or "rear-end shunt", identified by a British study as the most frequent cause of accidents leading to personal injury (Poston et al. 1983).

One factor that governs the queue psychology is a kind of "illusion of common movement" which is predicated upon the motorists' false perception that his spatial relationship with the line of vehicles ahead of him will be preserved. This illusion is compounded by the phenomenon of "expectancy" which predicts future events on the basis of recent past events. In this case the "expectancy" is the cars that have been moving along undisturbed for a number of miles, will continue unimpeded.

Platooning on a Rural Two-Lane Highway

A "platoon" exists when vehicles travel together as a group, either voluntarily or involuntarily because of signal control, geometrics, or other factors (Gattis et al. 1997). When vehicles are observed to be in platoons, it is quite possible that motorists behind the leading vehicle are being delayed. Such a state denotes a situation that motorists may perceive as reduced quality of transportation service.

With low volume of traffic on a given roadway, motorists seldom feel delayed and expect to be able to pass freely. As volumes approach capacity, the proportion of vehicles found in platoons increase, and delay increase. The Highway Capacity Manual (HCM) (Eyler et al. 1996) denotes that even though speeds may not drop to a crawl, "driver frustration would be excessive," if a high level of congestion routinely existed for a long periods of time.

The 1994 HCM states that on two-lane highways, "motorists are forced to adjust their individual travel speed as volume increases, and the ability to pass declines." HCM employs two traffic flow characteristics of average travel speed and percent time delay to define the level of service on two-lane rural highways. "Percent time delay" is the same as the percentage of time vehicles are delayed in platoons, with "delayed" defined as:

1. Travelling less than a desired speed
2. A headway of less than 5 sec.

For field measurement purposes, percent time delay is said to be "approximately the same as the percentage of all vehicles travelling in platoons at headways less than 5 sec." Thus, the HCM links road volume, driver delay, and vehicle headway. Even though spot platooning or percent following is defined in the HCM as the percentage of vehicles with headways of 5 sec or less as they pass a given point, different opinions exist concerning what headway time should be used to define delay. Messer and Morral (1983) used a platoon definition based on a 6-sec headway, and Hoban (1983) recommended a 4-sec headway criterion. Guell and Virkler (1995) claim that the use of 5 sec as the definition for delay produces an inconsistency in the level of service between certain general terrain segments and specific grades. Guell and Virker (1995) further state that the selection of 3.5 to 4.0 sec as the definition of delay would alleviate this problem. For example, at 97 kph (60 mph) the head-to-head spacing between two vehicles travelling at a 5-sec headway is 134 m (440 ft). This represents per-lane density of 7.5 vehicles per km (12 vehicles per mi). The value associated with level-of-service A would demand that the headway be less than the delay definition of 5 sec. At 4 and 3.5 sec, the spacings are 107 and 94 m (352 and 308 ft), respectively. Both of these conditions are associated with level-of-service B on a freeway (Mendoza et al. 1995).

Platoon Volume

Platooning varies with the volume of traffic. The number of platoons per hour and the number of vehicles in the platoons are functions of the total traffic volume. In an attempt to find the relationship between platoon volume and total volume, It was found that the number of vehicles in platoons per hour increases linearly with the total one-direction traffic volume (Gattis et al. 1997). A regression analysis on the data yielded the following relationship:

Number of vehicles in platoon/hr = -151+1.22x(total one directional volume)

Figure 3-2 represents this graphically. The R^2 value for the regression analysis was 0.97 with the independent variable ranging from 325 to 525 vph.

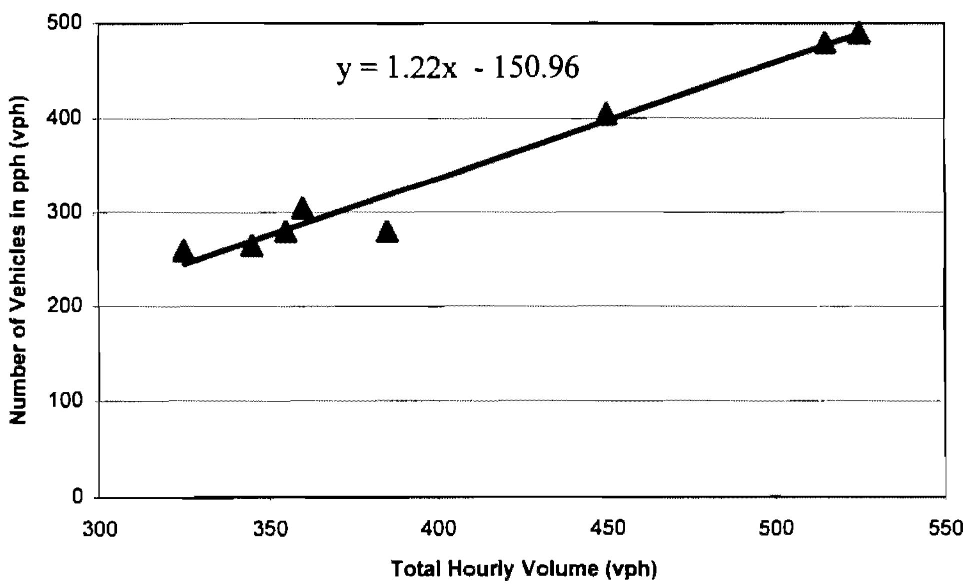


Figure 3-2: Number of Vehicles in Platoon v. Total Volume

Figure 3-3 represents the variation of platoons per hour as a function of total directional traffic volume.

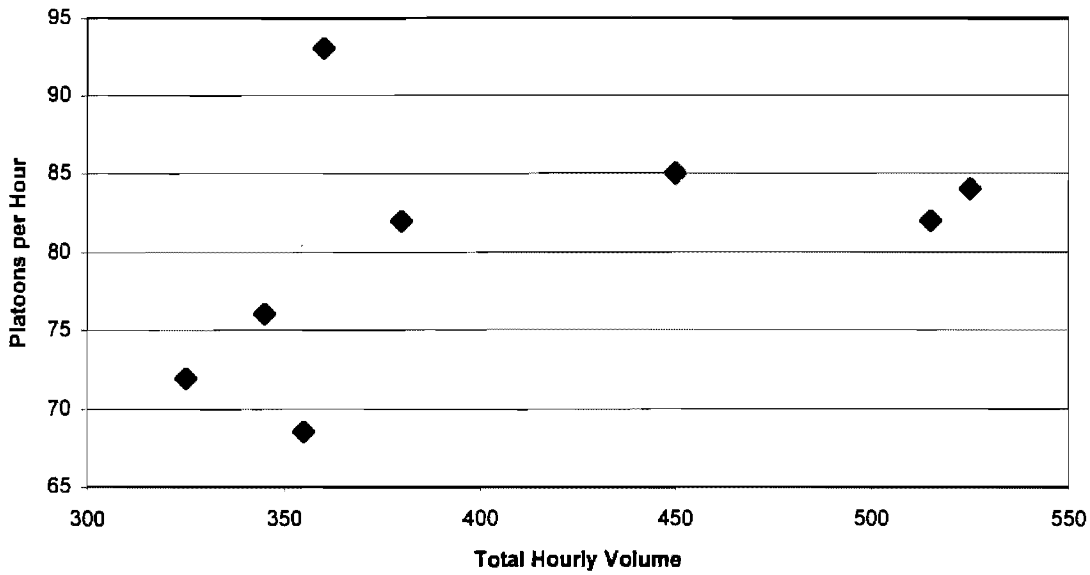


Figure 3-3 Number of Platoons v. Total Volume

From figure 3-3 the authors concluded that:

1. At low traffic volumes headways are high and the number of platoons per hour is relatively low.
2. As the traffic volume increases, more vehicles group together and the number of platoons per hour increases.
3. At very high volumes, smaller platoons merge to form larger platoons and the number of platoons tends to decrease.

Gap Acceptance Theory

The Super Two highways are modeled using the queuing theory as an analytical approach to the design of the passing lanes. The two main components of the queuing theory are the service and arrival concepts. A major implication in the application of the theory on a traffic problem is the dynamic characteristics of the traffic flow. The computation of the service rate (Q) should incorporate the fact that the truck percentage will have an influence on the service rate. Therefore, Q is computed from the equation that is published in the Highway Capacity Manual (1) on page 313. This equation gives the maximum service volumes on rural two-lane highways under uninterrupted flow conditions. The Highway Capacity Manual Table 10.7 (1) considers the truck factors (T_L) adjustment for lane width and lateral clearance (W_L) and a volume capacity ratio (v/c) depending on a passing site distance of 1500 ft which is considered a sufficient distance to perform a safe passing maneuver. Therefore, a probabilistic method considering the probability of having a safe passing distance in oncoming traffic is integrated to the service rate method. A decrease in the probability of a safe passing distance will give a lower volume capacity ratio (v/c), therefore yielding a lower service rate.

$SV=2000\ (v/c)\ W_L\ T_L$

Where SV= Service volume (mixed vehicles per hour, total for both directions)

v/c= Volume to capacity ratio, obtained from Highway Capacity Manual Table 10.8

W_L= Adjustment for lane width and lateral clearance at given level of service, obtained from Highway Capacity Manual Table 10.8

T_L= Truck factor at given level of service, obtained from Highway Capacity Manual Table 10.9b

In the equation above v/c factor is calculated from a table where the factor can be obtained for a corresponding probability of a passing sight distance greater than 1500 ft. Therefore, the gaps that will occur between the vehicles in oncoming traffic will have a direct effect on the service volume and the service rate consequently.

Physical conditions on the rural highway which is chosen for the case study is assumed to have 100% passing sight distance larger than 1500ft. However, as the ADT value increases passing sight distance provided by the opposing traffic will decrease. The cumulative probability of having a time gap that would allow a 1500ft passing sight distance between consecutive opposing vehicles are calculated. This probability is incorporated in the service volume equation, similar to the probability of having a physical passing sight distance of 1500ft. This approach enabled the dynamic behavior of service rate, because service rate or service volume decreased with increasing ADT levels.

The gap acceptance concept is used in probabilistic approach for the computation of the service rate Q. The equation below is adopted from Poisson distribution.

$$P(h \geq t)=e^{-\lambda t}$$

$$\lambda= V/T$$

λ is the average number of vehicles arriving per second

This equation gives the probability of having a gap of t seconds or greater in the traffic flow assuming a Poisson distribution. The outcome of the equation is used to determine the service volume of the highway by selecting a proper v/c ratio according to the probability of having an adequate passing distance from the table. This distance is given as 1500 ft. The maximum speed of the vehicles in oncoming traffic is 70 miles per hour. Accordingly, passing distance of 1500 ft gives a gap of 14.73 seconds. Then different service volumes are determined according the oncoming traffic. The oncoming traffic is forty percent of the total traffic flow assuming a directional split factor of 60/40.

Probability of available passing sight distance of 1500 ft during peak hour $P(h \geq t)$

The Probability distribution of opposing traffic is assumed to be *Poisson* distribution. The cumulative probability of having a time gap, that would provide a passing sight distance of 1500 ft, between two oncoming vehicles is found by Equation (1). Time gap (t) between two oncoming vehicles 1500 ft (1500 ft is taken to be equivalent to 450 m) apart and average number of cars in the opposing direction (λ) are calculated as follows.

$$t = \frac{(0.45)(3600)}{V_{pc}}$$

$$\lambda = \frac{(ADT)(PHF)(1-DF)}{3600}$$

$$P(h \geq t) = e^{-\lambda t} \tag{1}$$

where h = time gap between two consecutive opposing vehicles (sec)

t = time gap between two opposing vehicles 1500 ft apart (sec)

λ = average number of cars in the opposing direction (veh/sec)

PHF = Peak Hour Factor

DF = Directional Distribution Factor

ADT = Annual Daily Traffic

Summary and Discussion of the Results

Field observations and video records were made at three northwest Arkansas locations where drivers traveling on a two-lane rural highway encountered passing climbing-lane sections. At all three sites, vehicles had traversed roadway sections with limited passing opportunities before they encountered passing sections on slight to moderate upgrades. Two of the sites had relatively short passing lanes (<427 m.), and one lane had longer passing lanes (>762 m.). By studying traffic at such sites one can observe the behaviors of motorists who may have been restrained by slower traffic ahead, but then encounter a relatively unconstrained environment that allows them to pass the slower cars.

At higher traffic volumes (450 vph or higher), the arrival rate of vehicles in the samples taken probably did not follow a Poisson distribution. At lower traffic volumes (less than 400 vph), the arrival rates may have followed a Poisson distribution. As the flow rate increased, a model based on the assumption of random flow would not correctly reflect actual conditions at such sites. The number of vehicles in platoons per hour increased linearly with the total traffic volume. A regression analysis on the data yielded the following linear relationship:

number of vehicles in platoons/hr= -151 +1.22 x (total one direction volume)

The R^2 value for the regression analysis was 0.97 and the independent variable ranged from 325 to 525 vph.

The passing behavior studies used a radar gun to record vehicle speeds. The effect of the gun on driver behavior at these study sites is unknown. The concerns from the 1970s about the effects of radar gun use on data during the era of strict enforcement of the lower national speed limit would not be as relevant in the mid-1990s. A slightly smaller proportion of vehicles attempted to pass on the short lanes than on the long lanes. This could reflect driver judgment that there was insufficient distance in which to complete a pass on the short lane sections. In both data sets, passing success declined when headways were greater than 2.0 seconds.

Upon encountering a passing lane, drivers in vehicles behind the platoon leader are provided an opportunity to pass easily. There is more than one possibility as to why a driver would reject the opportunity. A driver declining the passing option may be resigned to an inadequate level of service or may not think that the effort to pass is justified. The primary explanation may be that the absence of passing when it is feasible indicates that the congestion, delay, and level of service are tolerable. At the short lane and the long lane sites when headways were 3.0 sec or more and platoon speeds were 50 mph or more, 85 percent of drivers exhibited little desire to pass. This suggests that many drivers readily tolerate a slight level of congestion or platooning on two-lane rural roads. The findings from this research support the views of those who consider the 5.0-sec headway to be excessive when defining delay on two-lane rural highways. A combination of both headway and platoon speed may more accurately define what the motorist considers to constitute delay.

Conclusions

After evaluating highway characteristics, the Super Two concept is a good approach to the solution of the problem. This type of highway with added passing lanes could provide a cost-effective method for improving the level of service on two-lane highways. Not only can passing lanes improve traffic operations on two-lane roads, but they have also been documented to reduce accidents. However, before the design phase a thorough analysis of the highway in terms of comparing savings with the construction and maintenance costs is important. In order to come up with a sound analysis, field data revealing the characteristics of a particular highway should be connected. By taking into account all delay and accident savings, Super Two should be considered a permanent solution or a temporary construction between two-lane highways and four-lane highways.

CHAPTER 4: EMERGING CONCLUSIONS AND RECCOMMENDATIONS

The most important conclusion is that Super Two has already been implemented in a number of states and at least five foreign countries. Therefore, the idea of providing a geometric bridge between two-lane and four-lane highways is indeed viable. In fact, it has already proven to be successful. The Minnesota DOT has already adopted design guidelines (Pitstic, 1991). In these guidelines the following statement is made:

"Super Two is a new type of roadway, ... The Super Two design should service relatively higher peak flows at a reasonable level of service by providing, as much as practical, an unimpeded traffic flow... In addition, construction of Super Two will have lesser environmental impact and right of way encroachment on adjoining lands than a four-lane expressway."

The need for an upgraded level of service on rural two-lane highways is further exacerbated by the anticipated increase in truck traffic due to the enactment of NAFTA. One corridor that is being proposed for traffic between Mexico and Canada is US Highway 83 (Morrall et al. 1992). Much of this road in Texas is rural two-lane highway. While ADT's will probably not greatly increase, the percentage of trucks with respect to total traffic will definitely go up. This creates a potentially dangerous situation. Implementation of Super Two on this corridor may provide a fast, low cost solution to this problem. By reducing the average delay time along a given route, new traffic is attracted to that route. This will result in increased economic development for the communities along that route. Thus, Super Two demonstrates the potential benefits beyond those that will be computed in this particular study.

The second conclusion is that time is the common parameter around which Super Two design geometry should be based. Time can be used to compute the optimum lengths of Super Two sections and the optimum distances between passing sections. The number of cars in a typical queue is a function of time and velocity. The average driver will willingly accept a certain amount of delay. However, once that time period has elapsed, natural impatience takes over and unsafe situations begin to develop. Therefore, if Super Two is to be successfully implemented, all its basic components must be designed around the acceptable delay time.

At this point, it is difficult to make many cogent recommendations. The study is at a point where there are few quantifiable results from which to base recommendations. As there are two Super Two pilot projects under design in the Childress District, the researchers will ensure that all emerging information is made available to the designers in that organization. Thus, the results of this study can be immediately rolled into the pilot projects giving TxDOT the ability to evaluate the recommendations of this study through field observation on actual highways. At this point we only wish to recommend that the classification system that has been devised be studied by Department personnel and considered for adoption, as the time becomes appropriate.

After evaluating highway characteristics the Super Two concept is a good approach to the solution of the problem. This type of highway with added passing lanes could provide a cost-effective method for improving the level of service on two-lane highways. Not only can passing lanes improve traffic operations on two-lane roads, but they have also been documented to reduce accidents. However, before the design phase, a thorough analysis of the highway in terms of comparing savings with the construction and maintenance costs is important. In order to come up with a sound analysis, field data revealing the characteristics of a particular highway should be studied. By taking into account all delay and accident savings, Super Two should be considered as a permanent solution or a temporary construction between two-lane highways and four-lane highways.

APPENDIX A: SHOULDER WIDTH IN LOW VOLUME ROAD DESIGN

NCHRP (1979) indicated that the primary functions of shoulders are to provide additional width for tracking corrections, head-on clearances, and emergency and leisure stops. Based on traffic data collected in New York by Billion (1959), this NCHRP report calculated the number of conflicts related to a stopped vehicle on a two-lane rural road without a shoulder (Table A-1).

Table A-1: Number of Conflicts Due to a Vehicle Parked in the Driving Lane on a Rural Two-Lane Roadway with No Shoulder (NCHRP, 1979)

ADT (vpd)	Number of Vehicles per Following Vehicle Arrival (per mile per day)	Number of Head-On Conflicts (per mile per day)
50	0.003858	0.0003
100	0.007716	0.0023
200	0.015432	0.0186
300	0.023148	0.0628
400	0.030864	0.1489

NCHRP (1979) also performed an analysis of road width requirements up to a design speed of 80km/hr for both tracking and head-on clearance. An extract of these requirements is given in Table A-2.

Table A-2: Total Road Width Requirement for Roads Based on Design Speed and Traffic Level (NCHRP, 1979)

Design Speed ,km/hr (mph)	Total Road Width Requirement, m (ft.)			
	Lower % Buses & Trucks <28% for 0-50 ADT <12% for 51-100 ADT <7% for 101-200 ADT <NA for 201-400 ADT		Higher % Buses & Trucks ≥28% for 0-50 ADT ≥12% for 51-100 ADT ≥7% for 101-200 ADT All % for 201-400 ADT	
	Infrequent Trips by Farm Machinery	Frequent Trips by Farm Machinery	Infrequent Trips by Farm Machinery	Frequent Trips by Farm Machinery
48 (30)	6.1 (20)	7.3 (24)	6.7 (22)	7.9 (26)
64 (40)	6.7 (22)	7.9 (26)	7.3 (24)	8.5 (28)
80 (50)	9.1 (30)	9.1 (30)	9.1 (30)	9.1 (30)

NCHRP (1979) suggested that it is justifiable to have shoulders if at least one critical conflict event takes place once every two weeks. Considering the analysis data presented in Table A-1, in the case of stopped vehicles, shoulders are justified for two-lane roads with ADT in excess of 320 vpd. Nevertheless, analysis data from Table A-2 suggest that shoulders two 3.6 m (12 ft.) lanes and two 0.9 m (3 ft.) shoulders are required for speeds as low as 80 km/hr (50 mph).

In 1982, another NCHRP report addressed the topic of *Shoulder Geometrics and Use Guidelines* and identified a number of design functions and uses of shoulders. Those design functions and uses of shoulders applicable to rural roads are listed below.

Design Functions:

1. Lateral support of main lanes
2. Delineation of travel way
3. Roadway drainage

Shoulder Uses:

1. Emergency stopping (mechanical difficulty)
2. Parking
3. Mail and other deliveries
4. Turning and/or parking at intersections
5. Routine maintenance activities
6. Snow storage
7. Arid areas
8. Major reconstruction and maintenance activities
9. Off-tracking
10. Encroachment
11. Slow moving vehicles
12. Pedestrians
13. Bicycles
14. Full running lanes
15. Errant vehicles
16. Emergency vehicle travel
17. Law enforcement
18. Emergency call box service
19. Roadside sales
20. Garbage pickup
21. Miscellaneous (funerals, snowmobiles, etc.)

These uses indicate that highway shoulders are used by the traveling public, adjacent property owners, agencies that build and maintain roads, and agencies that provide public services. This research project resulted in detailed guidelines for all the design needs and uses of shoulders identified therein. These guidelines included both acceptable and optimal values for the shoulder type (turf, gravel or paved), width, slope and strength. Table A-3 outlines the acceptable shoulder widths for various shoulder functions developed as a part of this study by NCHRP.

Table A-3: Acceptable Shoulder Widths for Various Shoulder Related Functions (NCHRP, 1982)

Shoulder Function	Appropriate Shoulder Width (ft.)
Roadway and shoulder drainage	1
Lateral support of pavement	1.5 to 2
Off-tracking of wide vehicles	2
Encroachment of wide vehicles	2
Errant vehicles	2
Mail and other deliveries	2
Pedestrians	4
Bicycles	4
Emergency stopping	6
Garbage pickup	6
Emergency vehicle travel	6
Routine maintenance	6
Law enforcement	6
Truck parking	10
Residential parking	7 to 8
Commercial parking	8 to 10
Turning and passing at intersections	9 to 10
Major reconstruction and maintenance	9
Slow-moving vehicles	9 to 10

In 1978, FHWA published proposed standards for roadway width. The roadway width was specified as a total roadway width, and not lane and shoulder widths separately. However, TRB (1987) published these FHWA standards as shown in Table A-4 so that they could be compared with the AASHTO standards. Later, TRB (1987) revised these standards as shown in Table A-5. TRB (1987) removed the "minor roads" criterion and also specified that the shoulder widths may be 1 ft. less than those specified in Table A-5 for mountainous terrain.

Table A-4: FHWA 1978 Roadway Width Standards

Current Traffic (ADT)	Design Speed (mph)	Width, ft			
		10% or More Trucks		Less than 10% Trucks	
		Lanes	Shoulders	Lanes	Shoulders
1-400	50 or Less	10	2	9*	2
	Over 50	10	2	10	2
401-4000	50 or Less	11	2	10	2
	Over 50	12	3	11	3
Over 4000	All	12	4	11	4

* - For minor roads only.

Table A-5: FHWA 1978 Roadway Width Standards Revised by TRB (TRB 1987)

Design Year Volume (ADT)	Average Running Speed (mph)	Width, ft			
		10% or More Trucks		Less than 10% Trucks	
		Lanes	Shoulders	Lanes	Shoulders
1-750	50 or Less	10	2	9	2
	Over 50	10	2	10	2
751-2000	50 or Less	11	2	10	2
	Over 50	12	3	11	3
Over 2000	All	12	6	11	6

In a 1987 study, Zegeer et al. analyzed accident data from 4951 miles of two-lane roadway in seven states and found that run-off-road, head-on collision and sideswipe (both same and opposite direction) to be the accident types most related to the roadway cross-section features. The roadway variables responsible for a reduced incidence of the above mentioned accident types were wider lanes, wider shoulders, better roadside condition, flatter terrain and lower traffic volume. Paved shoulders were found to be slightly safer than unpaved shoulders. For shoulder widths between 0 and 12 ft., the reduction in related accidents resulting from widening of paved shoulders was predicted to be 16 percent for a 2 ft. widening, 29 percent for a 4 ft. widening and 40 percent for a 6 ft. widening. Based on their work, Zegeer et al. (1987) developed a nomograph (Figure A-1) to predict paved and unpaved shoulder widths for seven roadside hazardous ratings they defined.

Griffin and Mak (1988) attempted to quantify the relationship between accident rate and roadway surface width of 36,215 miles of two-lane roads in Texas with ADT of 1500 or less. Their primary conclusions are given below.

1. Multi-vehicle accident rates are not related to surface width for any of the ADT groups tested.
2. For ADT between 401 and 1500, single vehicle accident rates increases with decreasing roadway width.
3. Economic analysis indicated that roadway widening is not cost-beneficial for ADT below 1000.

AASHTO (1990) defines a shoulder as a portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use, and for lateral support of subbase, base, and surface courses. In addition to the uses of shoulders as outlined in NCHRP (1982), AASHTO lists the following more important advantages of having well-designed and properly maintained shoulders.

1. The sense of openness created by shoulders of adequate width contributes much to driving ease and freedom from strain.
2. Sight distance is improved in cut sections, thereby improving safety.

3. Some types of shoulders enhance the esthetics of the highway.
4. Highway capacity is improved, and uniform speed is encouraged.
5. Lateral clearance is provided for signs and guardrails.
6. Storm water can be discharged farther from the pavement, and seepage adjacent to the pavement can be minimized reducing pavement breakup.
7. Improved lateral placement of vehicles and space for occasional encroachment of vehicles is provided.

AASHTO (1990) also stipulates that it is desirable that a vehicle stopped on the shoulder should clear the pavement edge by at least 1 ft., or preferable by 2 ft. For low-volume roads, AASHTO outlines the following policy.

1. Narrower shoulders are better than having none at all.
2. On low-volume highways, a minimum shoulder width of 2 ft. should be considered for the lowest type of highway and a 6 to 8 ft. width would be preferable.
3. Where roadside barriers, walls or other vertical elements are used, the graded shoulder should be wide enough such that these vertical elements can be offset by a minimum of 2 ft. from the edge of the usable shoulder.
4. Roadside barriers may be placed at the outer edge of the shoulder, with a minimum of 4 ft. from the traveled way to the barrier.
5. Regardless of width, shoulders should be continuous. Narrowing of shoulders may cause serious operating and safety problems. However, narrower and intermittent shoulders are superior to no shoulders.
6. Shoulders on structures should have the same width as the approach roadways.

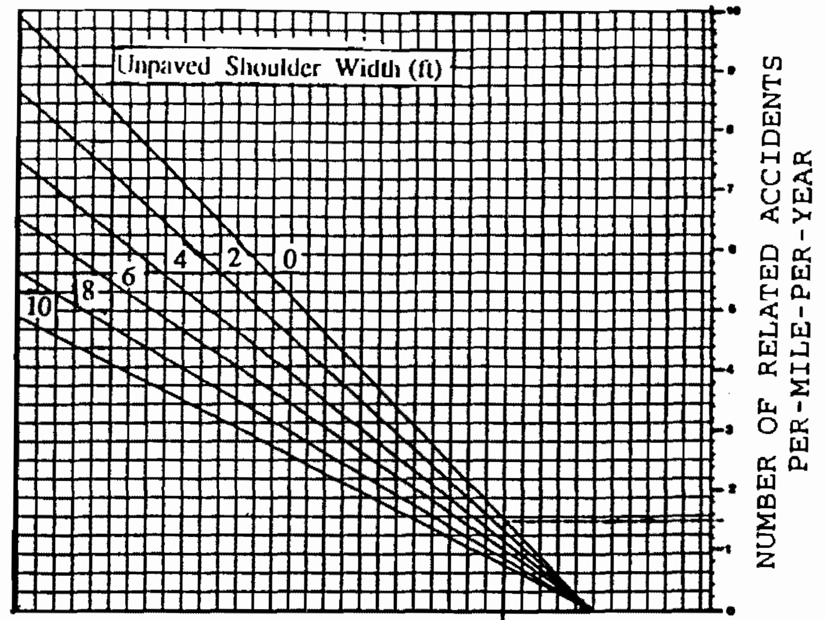
The AASHTO (1990) recommended shoulder widths for rural local, collector and arterial highways are given in Table A-6 below.

Table A-6: Recommended Shoulder Width for Rural Highways (AASHTO, 1990)

Type of Roadway	Shoulder Width, ft					
	ADT < 250	ADT 250-400	ADT > 400	DHV 100-200	DHV 200-400	DHV > 400
Rural Local Roadway	2	2	4	6	6	8
Rural Collector Roadway	2	2	4	6	8	8
Rural Arterial Roadway	4	4	6	6	8	8
Rural Local Bridge	2	2	3	3	3	Same as Approach Roadway
Rural Collector Bridge	2	2	3	3	4	Same as Approach Roadway

NCHRP (1994) made the following conclusions from a study that was conducted on roadway widths for low-volume roads.

1. Low-volume road accidents are affected primarily by roadway width, roadside hazard, terrain, and driveways per mile.
2. Accident rates are significantly associated with varying lane and shoulder widths for single vehicle and opposite direction accidents.
3. Presence of a shoulder is associated with significant accident reductions for lane widths of at least 10 ft.
4. For lane widths of 11 and 12 ft., shoulder widths of at least 3 ft. have significant effects.
5. There is no apparent accident reduction above a total roadway width of 30 ft.



Conditions for Use

1. Two-lane rural roads with an average daily traffic (ADT) of 100 to 10,000.

2. Lane widths of 8 to 12 feet.

3. Shoulders of 0 to 12 feet wide which are paved or unpaved.

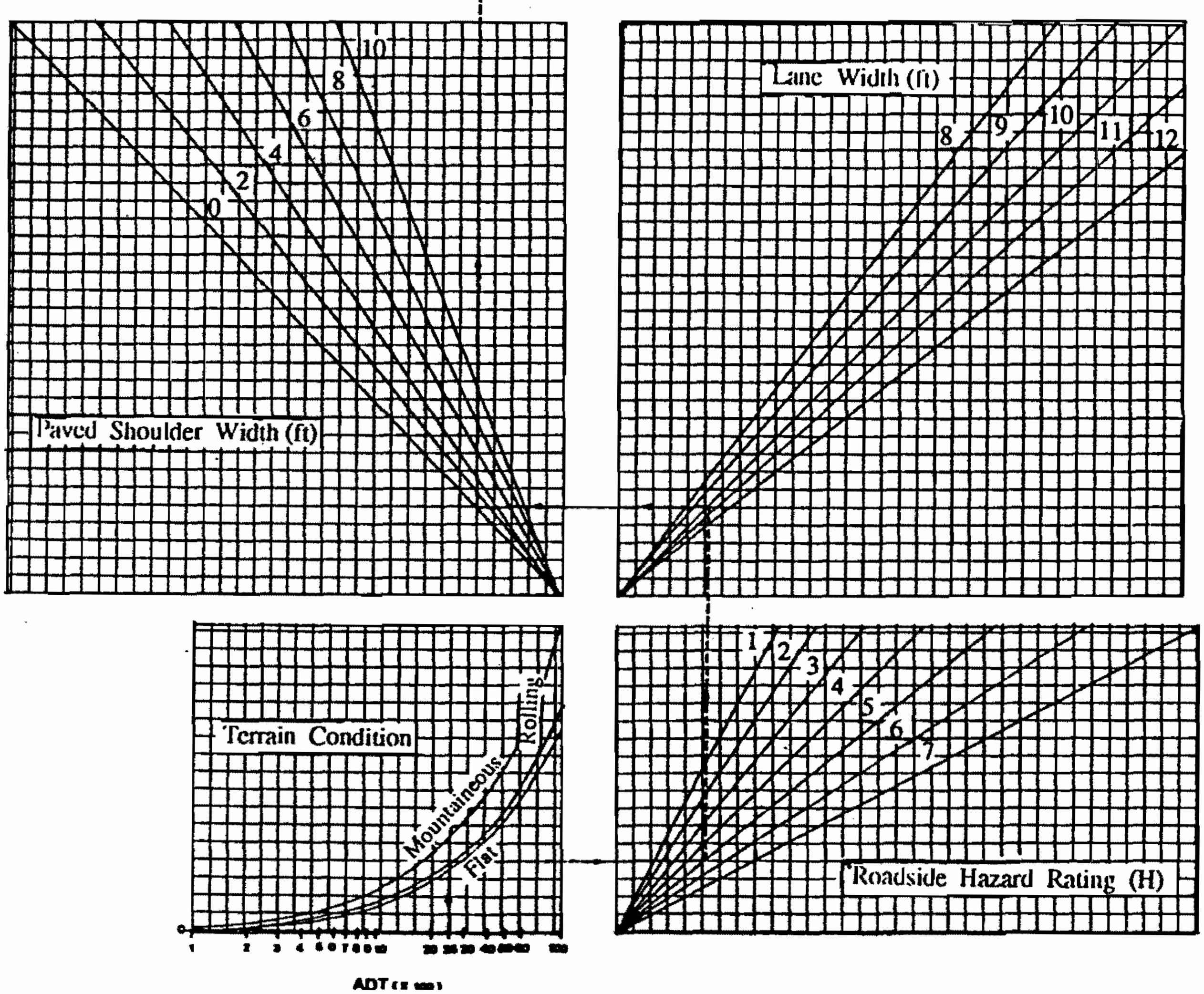


Figure A-1: Accident-predictive Nomograph (Zegeer et al. 1987)

APPENDIX B: SUPER TWO DESIGN ALTERNATIVES AND CLASSIFICATIONS

On two-lane rural roads, passing lanes have two important functions. One function is to reduce the delay at specific locations, such as steep upgrades or locations where trucks frequently travel next to farmland in the rural areas. The second function is to improve the overall traffic operations on a two-lane highway. This improvement can be made by breaking up the traffic platoons and reducing delays caused by inadequate passing opportunities over substantial lengths of highway. The design alternatives that have been evaluated for Super Two highways include many configurations of passing lanes. These alternatives can be classified into four types according to the passing lane configurations. These types are Type A, B, C, and D.

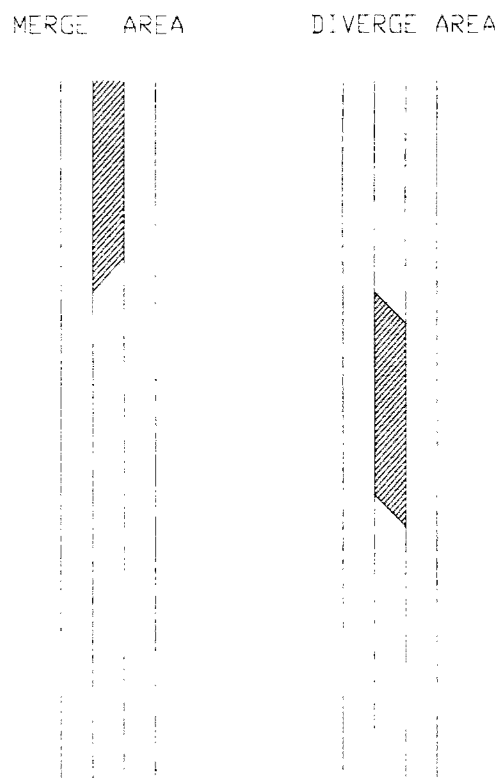


Figure B- 1: Type A Design with Continuous Interior Alternating Passing Lane

The basic Super Two section with alternate passing lanes in the middle provided in one direction or another. For that specific alternative the main advantage is there is no added cost of constructing an additional lane. This is true because the existing pavement width can be modified fulfilling the lane width and shoulder requirements. But the lane and shoulder widths may not be suitable for a roadway where traffic volume is even in both direction and where queuing occurs frequently in both directions.

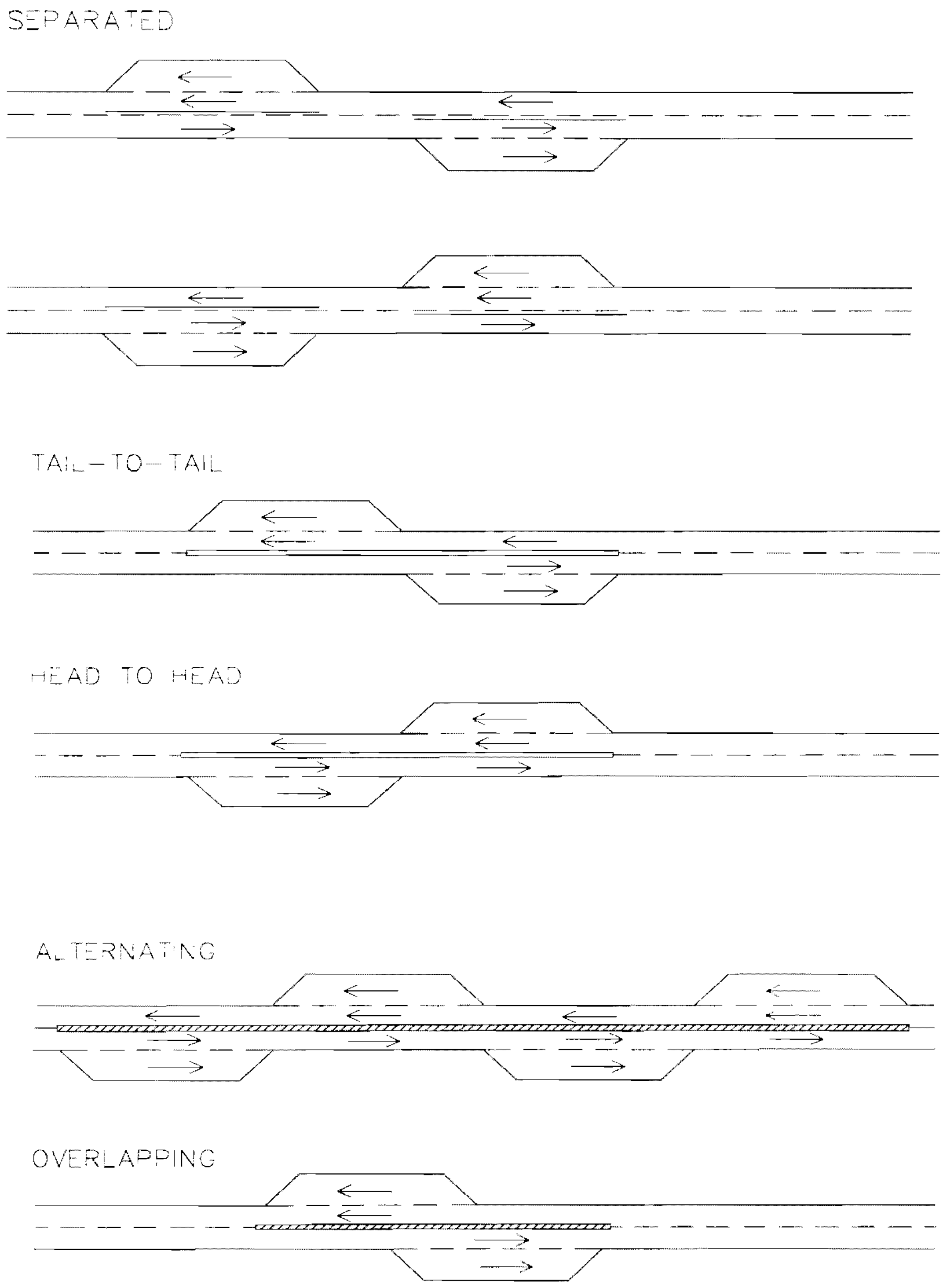


Figure B-2: Type B Design with Separated Passing Sections

Type B is the most common design used to provide passing lanes for the two-way two-lane rural highways. This is suitable for the highway sections having equal ADTs for both directions. The separated designs are often used in pairs, one in each direction at regular intervals along a two-lane highway. Where head-to-head or tail-to- tail sections are used passing by opposing direction vehicle is prohibited. The cost of constructing extra passing lanes should be considered before the final decision. The head-to-head and tail-to-tail sections will handle the platooning in different manners. The head-to-head configuration is preferable because the lane drop areas of the opposing passing lanes are not located adjacent to each other.

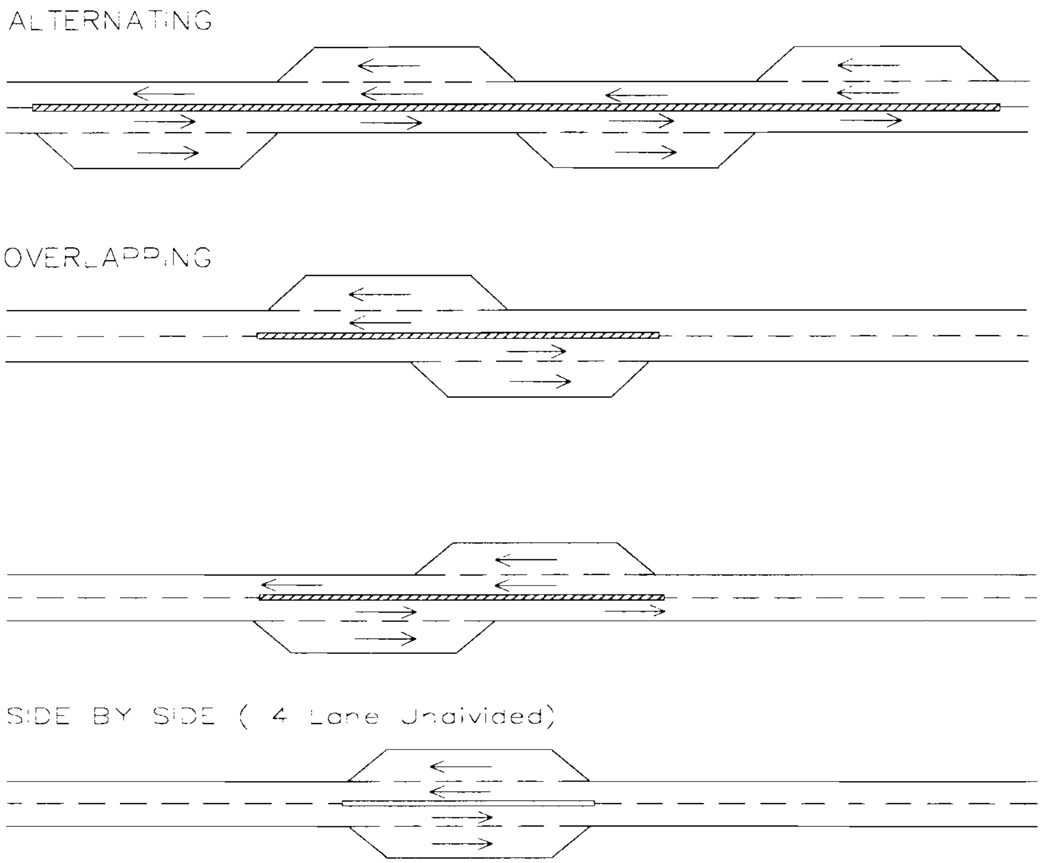


Figure B-3: Type C Design with Overlapping Passing Sections

The passing lane location should appear logical to the driver. It should provide passing facilities with a reasonable short delay time. Type C should be considered because of the specific need of a particular section of a rural two-lane two-way highway where the traffic volume is relatively high and similar in both directions. This kind of sections can be very effective close to the locality. Type C is a suitable alternative for future four-lane transformation of the rural highway.

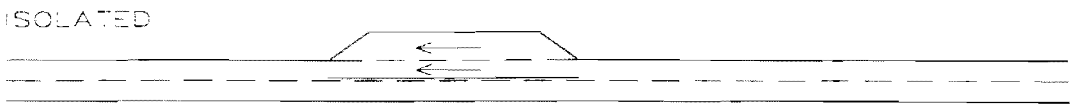


Figure B-4: Type D Design with Isolated Passing Section

This type of isolated passing lane can serve the specific purpose of reducing delay at a specific bottleneck. The importance may be noticed in an isolated section of a rural two-lane highway where passing opportunities require an isolated portion of the road. A vehicle using the rural road needs some sort of passing facilities where slow moving vehicles, like a cotton gin, might use the portion of the road frequently. One disadvantage of this kind of isolated passing lane is it would be less traveled because of the seasonal variation of traffic volume. However, from the economic point of view it's more cost effective than constructing passing lanes at regular intervals for the full length of the road.

APPENDIX C: APPLICATION OF QUEUING THEORY IN TWO-WAY, TWO LANE RURAL HIGHWAYS

The congestion of the traffic on urban highways especially during peak hours is a big concern of traffic engineers. This congestion results in the formation of queues on expressway on-ramps and off-ramps at signalized and unsignalized intersections, and on arterials where moving queues may occur. On rural highways the congestion of the traffic is much less frequent compared to the urban highways. However, the formation of the queue is still encountered on the rural highways. Slow moving vehicles preventing fast moving vehicles from driving at their desired speeds mainly cause that event. This is the case of a platoon of passenger vehicles being lead by a truck. An understanding of the processes that lead to the occurrence of queues and the subsequent delays on highways is essential for the proper analysis of the effects of queuing. The theory of queuing comprises the mathematical algorithms to describe the processes that result in the formation of the queues, so a detailed analysis of the effects of queues can be performed.

These mathematical algorithms can be used to determine the probability that an arrival will be delayed, the expected waiting time for all arrivals, the expected waiting time of an arrival that waits, and so forth. Examples where the theory can be applied include vehicles waiting to be served at a gasoline station, passengers or vehicles lined up at a transit terminal, and computer jobs waiting to print.

The service can be provided in a single channel or in several channels. Proper analysis of the effects of such a queue can be carried out only if the queue is fully specified. This requires the following characteristics of the queue be given (Garber et al. 1997).

- The characteristic distribution of arrivals, such as uniform and Poisson
- The methods of service, such as first come-first served, random and priority
- The characteristic of the queue length, that is, whether it is finite or infinite
- The distribution of service times
- The channel layout, that is, whether there are single or multiple channels and whether they are in series or parallel.

Some terms used to classify the queues are explained below.

Arrival Distribution: The arrivals can be described as either a deterministic distribution or a random distribution. A Poisson distribution usually describes light-to-medium traffic, and this is generally used in queuing theories related to traffic flow.

Service Method: Queues can also be classified by the method used in serving the arrivals. These include fist come fist serve, where units are served in order of their arrival, and last in-first served, where the service is reversed to the order of arrival. The service method can also be based on priority, where arrivals are directed to specific queues of appropriate priority levels. Queues are then serviced in order of their priority level.

Characteristics of the Queue Length: Queues can be classified as the maximum length of the queue, that is, the maximum number of units in the queue. Finite queues are sometimes necessary when the waiting area is limited.

Service distribution: This classification is usually considered random, and the Poisson and negative exponential distributions have been used.

Number of Channels: The number of the channels usually corresponds to the number of waiting lines and is therefore used to classify queues, for example a single-channel or multi-channel queue.

Oversaturated and Undersaturated Queues: Oversaturated queues are those in which the arrival rate is greater than the service rate, and undersaturated queues are those in which the arrival rate is less than the service rate. The length of an undersaturated queue may vary but will reach a steady state with the arrival of units. The length of an oversaturated queue, however, will never reach a steady state and will continue to increase with the arrival of units.

Single Channel, Undersaturated, Infinite Queues

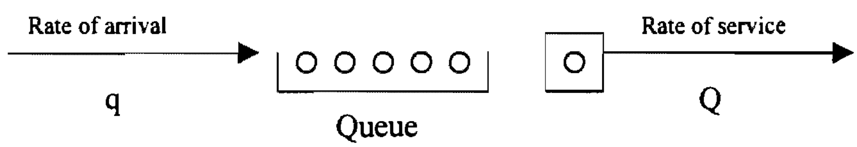


Figure C-1 Queue in a single channel

Figure C-1 represents a single channel queue where the service type is FIFO (first in first out) and with Poisson arrivals and exponentially distributed customer service times. The rate of arrival, q (vph), is less than the rate of service, Q (vph), since it is an undersaturated system. In the system, customers are assumed to be patient because they do not leave the system. This system is also assumed to have an unlimited holding capacity, which means there is no maximun limit on the number of customers that can be in the waiting line.

For an undersaturated queue (q<Q), assuming that both the rate of arrival and the rate of service are random, the following equations are developed. (Garber et al.1997)

1. Probability of n units in the system is:

$$P(n) = \left(\frac{q}{Q}\right)^n \left(1 - \frac{q}{Q}\right)$$

And n is the number of units in the system, including the unit being serviced.

2. The expected value, that is, the average number of units in the system at any time is:

$$E\,(n) = \frac{q}{Q-q}$$

3. The expected number of units waiting to be served (that is, the mean queue length) in the system is:

$$E\,(m) = \frac{q^2}{Q(Q-q)}$$

4. Average waiting time in the queue is:

$$E\,(w) = \frac{q}{Q(Q-q)}$$

5. Average waiting time in the system is:

$$E(v) = \frac{1}{Q-q}$$

Multi Channel, Undersaturated, Infinite Queues

A more complex queuing system is a FIFO system with N identical service counters in parallel. The average service rate per counter is q and Q is the arrival rate. The rate of service rate to the arrival rate is:

$$r = q/Q$$

In this case the probability equation is as follows. (Papacostas et al. 1993)

1. For n = 0:

$$p(0) = \left[\left(\sum_{n=0}^{N-1} \frac{r^n}{n!}\right) + \frac{r^N}{(N-1)!(n-r)}\right]^{-1}$$

For $1 \leq n \leq N$:

$$p(n) = \frac{r^n}{n!} p(0)$$

For n > N:

$$p(n) = \frac{r^n}{N! N^{n-N}} p(0)$$

2. The average number of units in the system is:

$$E(n) = r + \left[\frac{r^{N+1}}{(N-1)!(N-r)^2}\right]p(0)$$

3. The mean queue length is:

$$E(m) = \left[\frac{r^{N+1}}{(N-1)!(N-r)^2}\right]p(0)$$

4. The expected time in the queue is:

$$E(w) = \frac{E(m)}{q}$$

5. The expected time in the system is:

$$E(v) = \frac{E(n)}{q}$$

Application of the Theory

The major problem that creates the need for two-lane two-way highways to upgrade to Super Two Highways is the lack of passing opportunities. This event results in the time delay caused by speed differences between vehicles. In order to compute the average delay, the queuing theory is carried out in this research. The major components of the theory are rate of arrival and rate of service.

The Arrival Rate

The arrival rate is computed using ADT figures taken from the Amarillo District traffic map. On this map a particular section is considered for the calculation of the delay time. This specific section is on US Highway 87 at Dalhart in Dallam County (Northeast Texas). The ADTs range increased from 2400 to 3600 vehicles on that road. On rural roads with average fluctuation in traffic flow, the 30th highest hourly volume of the year approximates 15 percent of the ADT. The peak-hour of traffic is then equal to 15 percent of the ADTs. Accordingly, the arrival rates are calculated using the peak-hour traffic volume taking into account a directional distribution coefficient of 0.60 with an increment of 50 vehicle per hour.

The figure includes the numbers of cars and trucks traveling on the highway, on the other hand, queuing theory is modeled assuming the trucks are servers and the cars are units being served. In order to come up with a proper value to use in the performing of the queuing theory, the number of trucks should be extracted from the total number of vehicles. The deduction of 10 percent trucks gives the number of passenger cars used for calculating the arrival rate.

The Service Rate

Two different approaches for calculating the service rate are taken towards the solution of this problem. One is calculating the service rate from the service volume computed directly from capacity under ideal conditions. The procedure is as follows (Highway Research Board, 1987).

$SV = 2000\ (v/c)\ W_L\ T_L$

(v/c) = volume to capacity ratio

W_L = adjustment for lane width and lateral clearance at any given level of service

T_L = truck factor at given level of service

A typical cross section of a rural highway consists of two 12-foot lanes and two 10-foot shoulders making a 44-foot cross section. For our case we assume 10% truck traffic. The section that is considered has a maximum ADT of 3600 vehicles yielding a peak-hour traffic of 540 vehicles for both directions. This figure falls in the proper range (400-900) of service volume for Level of Service B. Therefore, the service rate is calculated for LOS B. As a result, the service rate is computed by using proper adjustment factors for a highway constructed on level terrain.

$SV_B = 2000\ (0.45)\ (1.0)\ (0.87) = 783$ vph, total for both directions.

Subsequently, the service rate is:

$Q = 783 \times 0.60 = 470$ vph = 7.83 veh/min, one direction

The sections without passing lanes are considered as a single channel model, the max arrival rate is

q_{max}= 4.86 veh/min (ADT=3600),

but the service rate can be considered without truck percentage that is 10%.

Q=7.05 veh/min is greater than q_{max}. As a result, the single channel undersaturated queue model is applicable for computing average delays per vehicle on that road.

Another way of determining the service rate is to find the number of cars that can pass a truck for a unit time. This method is useful since it reflects the real situation on the roadway from a physics standpoint. For that case, a physics problem where the period of time for a fast moving vehicle (passenger car) to overtake the slower moving vehicle (truck) is solved. The speed limit of 70 mph and 60 mph are the figures used in calculating the amount of time spent overtaking the slower moving vehicles leading the platoon. In order to perform a safe passing, for the speed group (60-70 mph) the distance traveled while the passing vehicle occupies the left lane (d_2) is 95.45 ft. The average passing speed for the same speed range is V_{avg}=62.38 mph (8.454 ft/sec). (AASHTO, 1994).

The computations are shown below.

$d_2 = (V_{avg})\ (t_{avg})$

$t_{avg} = 95.45 / 8.454 = 11.3$ sec/veh = 0.188 min/veh

By inverting the average time for a vehicle to pass the truck we can compute the average service rate Q_{avg}.

$Q_{avg} = 1 / t_{avg} = 5.32$ veh/min

<u>Sections with Passing Lane</u>

For the sections of the road with passing lanes, a multiple channel model with two identical channels (N=2) is used.

The average delay time per vehicle and the average queue length for sections with and without passing lanes are computed using the formulas given above. The results are given in a spreadsheet. There are two spreadsheets where two different service rates are evaluated according to the concepts discussed earlier.

Sample Problem:

For a Highway Section where an ADT of 2400 vehicles is observed, the truck percentage is 37%. Determine average delay time per vehicle.

Solution:

Peak-hour traffic: 2400 x 0.15 x 0.60 = 216 veh/hr (Total traffic for one direction)

216-216 x0.37 = 136 veh/hr (Number of cars-units served)

The arrival rate is: q = 2.27 veh/min

The service rate is: Q = 5.32 veh/min

Average delay: $E\ (w) = \dfrac{q}{Q(Q-q)} = \dfrac{2.27}{5.32(5.32-2.27)} x60 = 8.40 \sec /\text{veh}$

Note: Considering a traffic composition with 37% of trucks, the number of trucks is 80 meaning 80 platoons will form on the road. Then, the total delay time of a car to pass all the vehicles will be T_{tot} = 80 x 8.4 = 672 sec = 11.2 min.

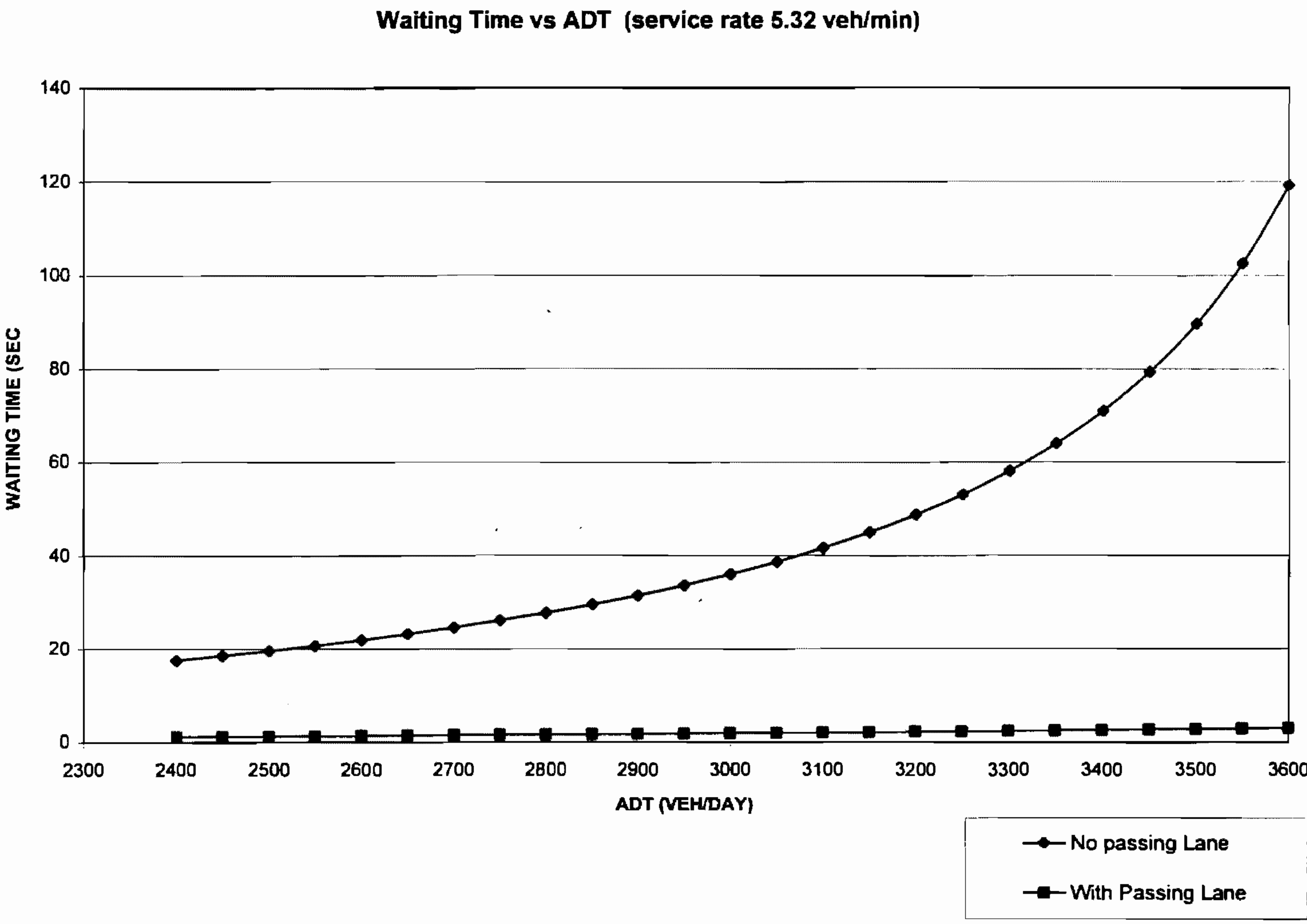
Conclusions

At this time the exact composition of the queue model is not fully determined. However, all of the elements have been assembled, and there appears to be no problem with the approach. Tables C-1 and C-2 illustrate the relationship between ADT and the average queue width. These tables prove the value of the Super Two concept. At the lower service rate, a queue will always form, but when important passing lanes are provided it can be emptied efficiently. For example, at 3600 ADT, there will be between 9 and 10 cars in the peak hour queue. But when provided a stretch of Super Two section, the mean queue length never exceeds 1 car. Thus, if the Super Two section is long enough the queue can empty. If the distance between sections is directly related to the time it takes to form the mean queue, the capacity of the Super Two section will be optimized with respect to traffic.

Table C-1

SERVICE RATE = 5.32 veh/min

ADT	PEAK	DIRECTIONAL	ARRIVAL RATE	Waiting Time - E(w) No passing Lane(sec)	Waiting Time - E(w) With Passing Lane (sec)	Mean Queue Length Single lane E(m)	Mean Queue Length Passing lane E(m)
2400	360	216	3.24	17.57	1.153	0.95	0.062
2450	368	221	3.31	18.54	1.206	1.02	0.067
2500	375	225	3.38	19.57	1.262	1.10	0.071
2550	383	230	3.44	20.68	1.319	1.19	0.076
2600	390	234	3.51	21.87	1.377	1.28	0.081
2650	398	239	3.58	23.16	1.438	1.38	0.086
2700	405	243	3.65	24.54	1.500	1.49	0.091
2750	413	248	3.71	26.05	1.563	1.61	0.097
2800	420	252	3.78	27.68	1.629	1.74	0.103
2850	428	257	3.85	29.47	1.697	1.89	0.109
2900	435	261	3.92	31.43	1.766	2.05	0.115
2950	443	266	3.98	33.58	1.837	2.23	0.122
3000	450	270	4.05	35.97	1.911	2.43	0.129
3050	458	275	4.12	38.62	1.986	2.65	0.136
3100	465	279	4.19	41.59	2.064	2.90	0.144
3150	473	284	4.25	44.93	2.144	3.18	0.152
3200	480	288	4.32	48.72	2.226	3.51	0.160
3250	488	293	4.39	53.06	2.311	3.88	0.169
3300	495	297	4.46	58.09	2.398	4.31	0.178
3350	503	302	4.52	63.96	2.487	4.82	0.187
3400	510	306	4.59	70.91	2.579	5.42	0.197
3450	518	311	4.66	79.29	2.673	6.15	0.208
3500	525	315	4.73	89.56	2.770	7.05	0.218
3550	533	320	4.79	102.47	2.870	8.18	0.229
3600	540	324	4.86	119.16	2.973	9.65	0.241



SERVICE RATE = 7.05 veh/min

Figure C-2

Table C-2

ADT	PEAK	DIRECTIONAL	ARRIVAL RATE	Waiting Time - E(w) No passing Lane(sec)	Waiting Time - E(w) With Passing Lane (sec)	Mean Queue Length Single lane E(m)	Mean Queue Length Passing lane E(m)
2400	360	216	3.24	7.24	0.474	0.39	0.026
2450	368	221	3.31	7.52	0.489	0.41	0.027
2500	375	225	3.38	7.72	0.510	0.43	0.029
2550	383	230	3.44	8.02	0.532	0.46	0.031
2600	390	234	3.51	8.33	0.555	0.49	0.032
2650	398	239	3.58	8.66	0.578	0.52	0.034
2700	405	243	3.65	8.99	0.601	0.55	0.037
2750	413	248	3.71	9.34	0.626	0.58	0.039
2800	420	252	3.78	9.71	0.650	0.61	0.041
2850	428	257	3.85	10.09	0.676	0.65	0.043
2900	435	261	3.92	10.48	0.701	0.68	0.046
2950	443	266	3.98	10.90	0.728	0.72	0.048
3000	450	270	4.05	11.33	0.755	0.76	0.051
3050	458	275	4.12	11.78	0.783	0.81	0.054
3100	465	279	4.19	12.25	0.811	0.85	0.057
3150	473	284	4.25	12.75	0.840	0.90	0.060
3200	480	288	4.32	13.26	0.870	0.96	0.063
3250	488	293	4.39	13.81	0.900	1.01	0.066
3300	495	297	4.46	14.38	0.931	1.07	0.069
3350	503	302	4.52	14.99	0.963	1.13	0.073
3400	510	306	4.59	15.62	0.995	1.20	0.076
3450	518	311	4.66	16.29	1.028	1.26	0.080
3500	525	315	4.73	17.00	1.062	1.34	0.084
3550	533	320	4.79	17.75	1.096	1.42	0.088
3600	540	324	4.86	18.55	1.132	1.50	0.092

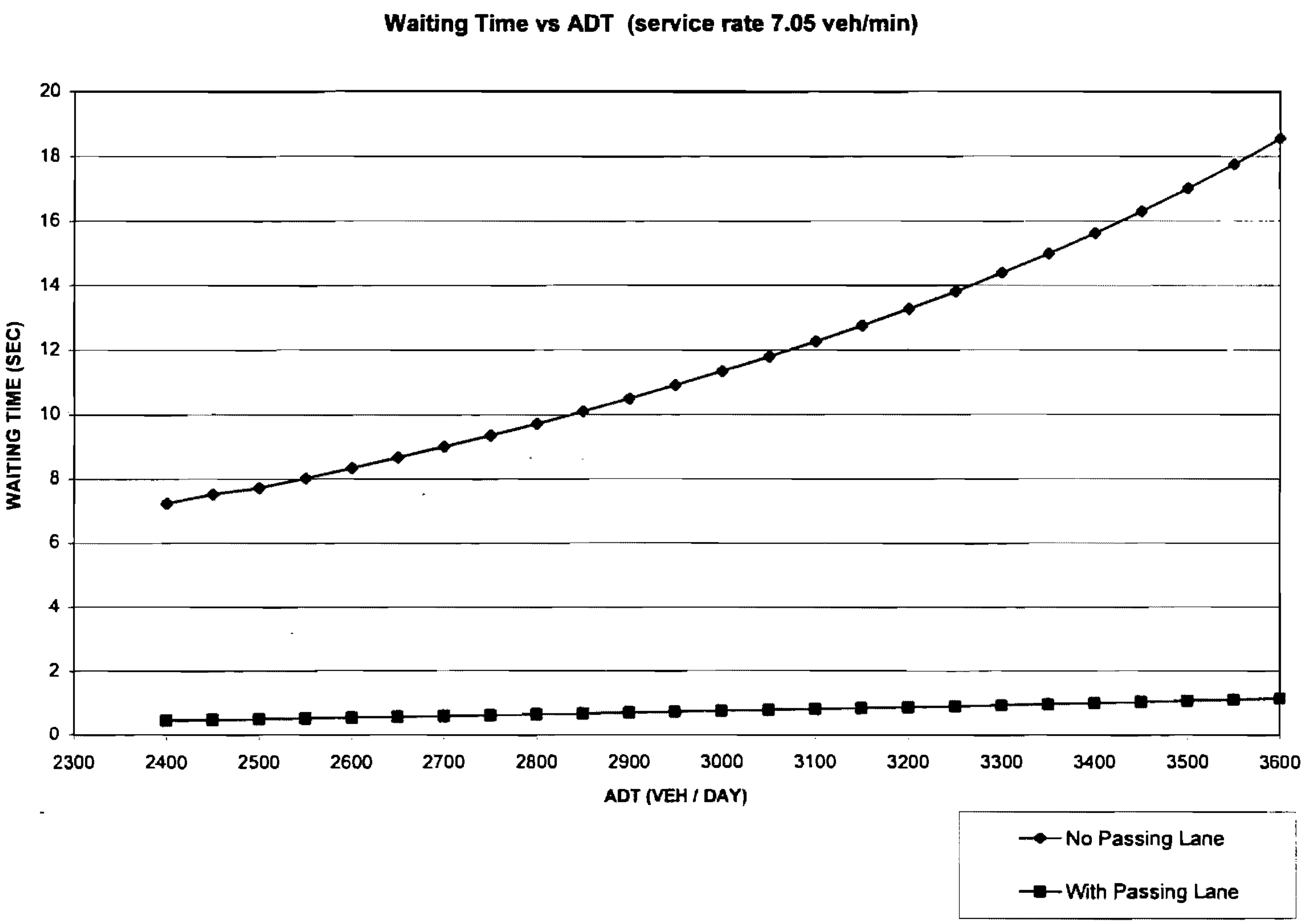

Waiting Time vs ADT (service rate 7.05 veh/min)
WAITING TIME (SEC)
20
18
16
14
12
10
8
6
4
2
0
2300
2400
2500
2600
2700
2800
2900
3000
3100
3200
3300
3400
3500
3600
ADT (VEH / DAY)
No Passing Lane
With Passing Lane

Figure C-3

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